Challenges and Potentials of Retrofitting Masonry Non-Engineered Construction in Indonesia

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This dissertation is written in memory of the many casualties caused by the collapse of non-engineered construction during past earthquakes in Indonesia and with the hope that in the future there will be no more casualties.

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Executive Summary

Indonesian non-engineered construction mostly consists of masonry structures confined and / or unconfined. The construction of masonry buildings are not too complicated, therefore, it is widely used all over Indonesia. It is also known that masonry is brittle and unless provided with reinforcement, or other suitable materials, such buildings are weak against earthquakes.

With the extreme pressures of a great demand for new masonry houses together with a limitation on the resources available, including finance, skills, and building materials, resulting in poor workmanship and poor quality of construction. The general tendency has been for the standards to fall year by year. World experience in damaging earthquakes has shown that these types of construction are dangerous to human life, often in a relatively small earthquake. It is quite apparent that it will be difficult to do away with this kind of construction in seismic areas, particularly in developing countries, because brick is relatively cheap, easy to produce and to transport, and masonry construction is relatively easy to construct. Those factors have made masonry very suitable as a construction material, and therefore, the trend to build more and more masonry buildings is obvious.

The objective of the study is to find some means and methods to improve the present construction and the related materials using available materials with local labor under minimal supervision and most suitable to the local culture, particularly in Indonesia. The main aim is to save human life; therefore, structures might be damaged when shaken by earthquakes, but does not collapse and kill people.

In general, this dissertation can be divided into three main parts.

The first part of the dissertation contains explanation why Indonesia continuously still experiences damages of non-engineered construction in spite of the fact that considerable research and available guidelines regarding non-engineered construction were available since 35 years ago. Almost every year earthquake disaster occurs in many places in different parts of Indonesia and causes damage and destruction to non-engineered constructions. Despite of the many human casualties and the severe impact on the regional economy and development, it seems that relatively little is being done to prepare, prevent or mitigate the effects of future earthquakes. The earthquakes are repetitions of all past occurrences and are demonstrations that not much has been done with regard to non-engineered constructions. With the re-occurrence of the same mistakes until today, “the earthquake problem in Indonesia“ should be reviewed so that the necessary action can be taken to prevent damage and casualties in future earthquakes. Two major issues related to non-engineered construction in Indonesia will be discussed: the unsafe non-engineered construction stock in Indonesia; and ineffectiveness of disaster risk reduction. It is evident that the number of non-engineered constructions in Indonesia that are not earthquake resistant is increasing year by year and the understanding and realization of disaster risk reduction in Indonesia is limited if not none. This is the real Indonesian earthquake problem that must be resolved using simple and affordable methods.

The second part describes the non-engineered construction in Indonesia, the damages of non-engineered constructions from the past 40-years earthquakes, the causes of damages, the problems encountered in implementing the earthquake resistant non-engineered constructions, and design basis of non-engineered constructions. The methodology used is observation survey.

Typical Indonesian non-engineered construction consists of unconfined and confined masonry. The unconfined masonry buildings were introduced by the Dutch when Indonesia was a colony of the Dutch hundreds of years ago. This type of masonry buildings is copied from Europe and consists of one brick thick walls, using brick pilasters without any reinforced concrete columns and beams as confinement. Trass lime blocks, concrete hollow blocks were introduced in the 60's.

After Indonesia becomes an independent nation, the demand for masonry buildings / houses is substantial and due to the increase in cost, people started building half-brick masonry houses. In the very beginning, those half-brick masonry buildings / houses were built without any reinforcement, the so called Unreinforced Masonry (URM). However, from documenting earthquake damages in various areas in Indonesia over the past 40 years, it is evident that in almost all rural as well as urban areas all over Indonesia, a good earthquake resistant design feature can be identified, namely almost all half-brick-thick masonry buildings are built with reinforced concrete framing, consisting of the so called "practical columns and beams" (Boen, 2006), forming confined masonry walls. In some places in Indonesia, timber is also used as framing to confine the masonry walls. In addition, it is also found the use of bamboo as replacement of reinforcing bars in "practical columns and beams".

The confined masonry construction using reinforced concrete framing has become a new culture all over Indonesia and from past earthquakes it is evident that provided they are built with good quality materials and good workmanship, they can survive the most probable strongest earthquake in accordance with the Indonesian seismic hazard map (Boen, 2003; Boen, 2006; Boen, 2007; Boen, 2007). Shaking table tests that were performed in Japan showed good results. However, due to poor quality of materials and poor workmanship, resulting in, among others poor detailing, poor mortar quality, poor concrete quality, and poor brick-laying, this masonry construction became not resistant to earthquakes and could be damaged and even collapsed when shaken even by minor earthquakes. In general, the quality of workmanship for the newly constructed houses in Indonesia is below average and in many cases poor. This is clearly demonstrated in the reconstruction of Aceh, after the 2004 tsunami (Boen, 2006). Poor quality materials (such as bricks, sand, and timber) combined with poor workmanship (Boen, 2006; Boen & Priyono, 2011) and non-compliance with the Indonesian seismic guidelines resulted in many houses reconstructed so far are below standard. In many instances, the "quality" is enhanced a bit due to the widely use of Portland Cement mortar.

Surveys and tests of building materials were conducted in recent years by Universities, and foreign government agencies in several places in Indonesia. The objective of these surveys is to know the quality of local building materials as well as workmanship. The survey and test results showed that there are many variations of brick dimensions. The qualities of brick-works also vary, from good enough until poor brick-work. From site observations, it was also evident that many of the masons as well as carpenters are "instant" masons and carpenters and lack the necessary skills and apply incorrect mixing of mortar as well as concrete. This can be observed from the results of their works. Reconstruction of 127,400 houses in Aceh is evidence that in general, the quality of workmanship is below average and in many cases poor (Boen, 2006).

Learning from past earthquake damages, typical damages of non-engineered constructions can be identified. With the increased computing power and speed of desktop / laptop computers and also the availability of softwares, particularly in the last 15 years, static and dynamic analysis of structures can be quickly and efficiently performed by the engineers (Boen, 2001; Boen, 2003; Boen, 2007). The purpose of the analysis is not to simulate the actual behavior, but to get reliable information that there is a correlation between the observed damages and the results of the analysis. The correlation is not perfect, but is good enough to get a good idea to build appropriate non-engineered constructions that can withstand earthquakes (Powell, 2013).

The third part of the dissertation contains the proposed retrofitting method that is simple, affordable and replicable, for existing non-engineered constructions in Indonesia. **The method proposed is not scientific brain teasing research stuff, but an engineering design utilizing all existing references on the subject, meaning not "re-inventing the wheel". The methodology used is taken from literature study and make use of existing theories regarding the proposed method of retrofitting.** Besides literature study, shaking table test was performed in Japan, for Indonesian types of non-engineered constructions retrofitted with wire mesh.

As mentioned in part one, **millions of non-engineered constructions in Indonesia are vulnerable and a simple, affordable and replicable method to strengthen the existing non-engineered construction in Indonesia is introduced.**

The principle of sandwich structures will be introduced because the same principle of sandwich structures can be applied to strengthen unreinforced masonry walls, i.e. brick wall as core and ferrocement as skin facings. The analysis and design will be explained and subsequently an example of the analysis and design utilizing existing commercial software will be performed.

There are few researchers that mentioned one of the retrofitting methods of walls using ferrocement using welded mesh located at the center of the ferrocement layer. However all papers (ElGawady, et al., 2004; Muntean, et al., 2010) were mostly laboratory tests and did not explain in detail how to implement it. Apart from that, those papers also did not explain how to analyze the wall and DID NOT CORELATE IT as a sandwich structure, in which brick-walls act as a core and ferrocement on both sides of the walls act as skin facings (ElGawady, et al., 2004). Another paper deals with similar strengthening URM, also using ferrocement, however, sandwich analogy was not applied (Muntean, et al., 2010).

In 2012 a full-scale shaking table test was conducted in Japan. The results of the test will be highlighted to confirm the soundness of the proposed retrofitting method. The methodology used is experimental and verified by analysis.

The closing chapter will explain the way forward how to improve disaster risk reduction in Indonesia and the applicability of the proposed retrofitting method for other countries that have similar masonry construction.

Chapter 1 Introduction

1.1. Background

Almost every year earthquake disasters occur in various areas in Indonesia and despite of the severe impact on the regional economy and development, it seems that relatively little is being done to prepare for, prevent or mitigate the effects of future earthquakes.

Throughout the centuries, earthquakes have taken a high toll of human lives and caused great property losses throughout the world and unfortunately mostly in developing countries. All the catastrophes are due to the collapse of man-made buildings/structures.

In general, buildings can be divided into two main categories, namely engineered buildings and non-engineered constructions, their percentages being quite different in developed, developing, and underdeveloped countries. Past destructive earthquakes showed that most of the disasters occurred to non-engineered constructions. In Indonesia, most dwellings (non-engineered constructions) constructed in small towns and villages are built according to tradition, their types suiting the culture and materials available in that area. The traditional houses generally have a good record or performance in past earthquakes. However, as the economic condition is prospering, there is a strong trend towards the construction of masonry houses and measure of status is associated with the owners of such masonry houses. Poor people tend to adopt such new habits and built "look like masonry" houses. Most of such masonry houses are built without considering the requirements for appropriate masonry construction (Boen, 1978).

With the extreme pressures of a great demand for new masonry houses together with a limitation on the resources available, including finance, skills, and building materials, resulting in poor workmanship and poor quality of construction. The general tendency has been for the standards to fall year by year. World experience in damaging earthquakes has shown that these types of construction are dangerous to human life, often in a relatively small earthquake. It is quite apparent that it will be difficult to do away with this kind of construction in seismic areas, particularly in developing countries (Boen, 1978).

All of the damages to date are repetitions of all past occurrences and are a demonstration that in Indonesia not much has been done with regard to non-engineered constructions. Judging from the list of destructive earthquakes dated back in 1821 as can be seen in Appendix A, although qualitative, it was reported that many buildings were damaged or collapsed. Obviously all those buildings must be non-engineered constructions, judging that engineered buildings were only constructed some 100 years ago only. Apart from that list, from the author's surveys and documenting 49 destructive earthquakes as listed in Table 1, almost all the damaged buildings were non-engineered constructions. Even the July 2, 2013 earthquake in Aceh Tengah and Bener Meriah in Aceh Province also showed that many non-engineered houses were heavily damaged and / or collapsed.

Table 1 – List of 49 Destructive Earthquakes Surveyed by the Author

No	Earthquake Locations	Local Date	Local Time	Epicenter	Depth (km)	Magnitude
1	Sumbawa	2-Nov-54	16:24:54	8.0°S, 119.0°E	N/A	6.75
2	Aceh	2-Apr-64	8:11:55	5.90°N, 95.70°E	132	5.2
3	Bali	14-Jul-76	15:13:22	8.2°S, 114.9°E	N/A	6.2
4	Pasaman	9-Mar-77	06:17:29	0.4°N, 99.7°E	N/A	6.0
5	Bali-Lombok-Sumbawa	20-Aug-77	03:06:08	11.09°S, 118.46°E	33	7.0
6	Mataram-Lombok	30-May-79	17:38:52.9	8.21°S, 115.95°E	25	6.1
7	Tasik-Garut	2-Nov-79	22:53:03	8.6°S, 107.8°E	64	6.4
8	Manado	22-Feb-80	11:51:46	1.5°N, 124.65°E	N/A	5.5
9	Sukabumi	10-Feb-82	16:17:51.5	7°S, 106.94°E	25	5.3
10	Flores	25-Dec-82	20:28:02.8	8.41°S, 123.08°E	33	5.9
11	Aceh	4-Apr-83	09:51:13.9	5.8°S, 93.27°E	51	6.6
12	Tarutung	26-Apr-87	02:22:07.2	2.244°S, 98.866°E	11	5.9
13	Majalengka	6-Jul-90	07:16:20.4	6.904°S, 108.120°E	14	5.8
14	Flores	12-Dec-92	13:29:26.3	8.480°S, 121.896°E	28	7.8
15	Halmahera	21-Jan-94	11:24:29.9	10.15°S, 127.733°E	20	6.2
16	Liwa	16-Feb-94	0:07:43.8	5°S, 104.3°E	33	6.3
17	Banyuwangi	4-Jun-94	04:06:59.8	10.362°S, 112.892°E	26	7.8
18	Serui	21-Nov-94	01:59:06.4	2.001°S, 135.931°E	24	5.7
19	Pulau Obi	28-Jan-95	05:16:52.1	4.434°S, 134.476°E	22	6.2
20	Dili	14-May-95	19:33:18.8	8.378°S, 125.127°E	11	6.2
21	Palu	20-May-95	05:30:06.4	1.021°S, 120.505°E	26	5.5
22	Kerinci	7-Oct-95	1:09:45.9	2.1°S, 101.3°E	33	7.0
23	Biak	17-Feb-96	23:21:22.3	0.567°S, 135.840°E	19	8.1
24	Pare-pare - Pinrang	28-Sep-97	8:38:28.6	3.91°S, 119.7°E	33	6.0
25	Pandegelang	22-Dec-99	21:14:57.6	7.21°S, 105.64°E	25	6.0
26	Banggai	4-May-00	11:21:16.2	1.65°S, 123.79°E	68	6.5
27	Bengkulu	4-Jun-00	23:28:26.1	4.7°S, 102°E	33	7.3
28	Manokwari	10-Oct-02	21:28:25.8	1.511°S, 133.973°E	10	6.7
29	Karangasem	2-Jan-04	03:59:31.9	8.310°S, 115.788°E	45	5.8
30	Nabire	6-Feb-04	06:05:02.8	3.615°S, 135.538°E	17	7.0
31	Padang Panjang	16-Feb-04	21:44:39.9	0.466°S, 100.655°E	56	5.1
32	Alor	12-Nov-04	05:26:41.1	8.152°S, 124.868°E	10	7.5
33	Nabire	26-Nov-04	11:25:03.3	3.609°S, 135.404°E	10	7.1
34	Aceh	26-Dec-04	07:58:53.4	3.295°N, 95.982°E	30	9.0
35	Palu	24-Jan-05	04:10:12.1	1.198°S, 119.933°E	11	6.3
36	Nias & Simeulue	28-Mar-05	23:09:36	2.08°N, 97.01°E	30	8.7
37	Yogyakarta	27-May-06	05:53:58.9	7.961°S, 110.446°E	13	6.3
38	Pangandaran	17-Jul-06	15:19:26.6	9.284°S, 107.419°E	20	7.7
39	North Sumatra	18-Dec-06	04:39:17.4	0.626°N, 99.859°E	30	5.8
40	West Sumatra (Solok-Padang)	6-Mar-07	10:49:38.9	0.493°S, 100.498°E	19	6.4
41	Bengkulu	12-Sep-07	18:10:26.8	4.438°S, 101.367°E	34	8.5
42	Sumbawa (Dompu)	26-Nov-07	00:02:15.7	8.292°S, 118.370°E	20	6.5

No	Earthquake Locations	Local Date	Local Time	Epicenter	Depth (km)	Magnitude
43	Simeulue	20-Feb-08	15:08:30.5	2.768°N, 95.964°E	26	7.4
44	Manokwari	4-Jan-09	07:33:40.2	0.691°S, 133.305°E	23	7.4
45	West Java	2-Sep-09	14:55:01.0	7.782°S, 107.297°E	46	7.0
46	West Sumatra	30-Sep-09	17:16:09.2	0.720°S, 99.867°E	81	7.6
47	Simeulue	7-Apr-10	05:15:01.5	2.383°N, 97.048°E	31	7.8
48	Mentawai	25-Oct-10	21:42:22.4	3.487°S, 100.082°E	20	7.8
49	Aceh	2-Jul-13	14:37:02	4.698°N, 96.687°E	10	6.1

When the Dutch occupied Indonesia, they introduced "modern" type buildings; among others masonry and timber constructions, people start copying the Dutch introduced types of buildings. Since Holland is relatively free of earthquakes, during that time the Dutch were not familiar with earthquake resistant construction and therefore, they never constructed earthquake resistant buildings. However, the quality of construction during the Dutch occupation was relatively much better than after Indonesia became an independent country in 1945.

In 1978 the author already observed and wrote in the manual (Boen, 1978) that the building discipline in Indonesia has gone down and the quality of materials used as well as the quality of workmanship for non-engineered constructions is very poor. This was also confirmed through JICA studies (Building Research Institute, 2006; JICA Manado Survey Team, 2009; JICA - Jurusan Teknik Sipil Universitas Negeri Padang, 2009; JICA - Jurusan Teknik Sipil Universitas Negeri Padang, 2010).

From the above explanations and the reoccurrence of damaged and collapsed of non-engineered constructions after every earthquake so far, it can be concluded that in Indonesia millions of non-engineered constructions are not earthquake resistant. From an economic point of view, it is unreasonable to rebuild all the structures that cannot withstand earthquakes, although such an action is ideal. It should be carefully decided that whether to rebuild or merely strengthen.

Retrofitting is the strengthening of buildings to increase the earthquake resistance. It is an intervention that preferably is implemented as an integrated program covering entire settlement areas. It involves public measures of support and promotion of retrofitting programs, but above all requires a high degree of dwellers participation (Nimpuno, 1992).

Retrofit is done to improve the seismic safety of existing buildings:

- Retrofit of existing non-engineered constructions: data from the Biro Pusat Statistik (BPS) shows in 2010 there are approximately 30,218,454 houses in urban and 30,887,004 houses in rural area at 33 provinces in Indonesia. Most of those houses, particularly in rural areas and urban informal settlements, are susceptible to earthquakes and should be assessed whether the buildings have met earthquake resistant requirements.
- Retrofit buildings damaged during earthquakes: right after an earthquake, it is common that people are in doubt to determine which buildings should be demolished, which ones must be repaired, and which ones must be strengthened, and how to do it. Many damaged buildings were demolished since the owners were

not aware that their buildings do not have to be demolished. The retrofitting cost is much less than the cost to build a new one. Moreover, the time required to retrofit is also shorter compared to building new ones. Figure 1 shows example of houses that can be retrofitted. This is very true and valid for one- or two-story school buildings which can be categorized as non-engineered.

- Retrofit to comply with new codes (note: if codes for non-engineered construction already exist): Design of new buildings for earthquake resistance is a relatively recent development. Provisions for seismic design and detailing of members and structures resembling those found in modern seismic codes did not appear before mid-1970s in any standard in the world. The building inventory of many seismic regions worldwide is by and large substandard and seismically deficient. Although today and for the years to come the major earthquake threat to human life and property comes from existing substandard buildings, the emphasis of earthquake engineering research, practice and code-writing has been, and still is, on new construction. Seismic retrofitting of buildings is effective in mitigating the seismic risk posed by a substandard building stock. The need to retrofit or not for a specific building and the scope and targets of the retrofitting normally comes out of a detailed seismic assessment or evaluation of the building to resist design earthquake forces based on current building codes (Fardis, 2009). If the assessment found that structures are not adequate, retrofitting should be introduced to improve the building's performance.

Figure 1 – Example of Houses that can be Retrofitted

Figure 1 (cont'd) – Example of Houses that can be Retrofitted

Figure 1 (cont'd) – Example of Houses that can be Retrofitted

1.2. Objective

The objective of the study is to find some means and methods to improve the present non-engineered construction and the related materials. The main aim is to save human life; therefore, structures might be damaged but does not collapse and kill people.

This dissertation is not a brain teasing scientific research stuff, but an engineering design utilizing all existing references on the subject, meaning not "re-inventing the wheel".

Academics tend to solve the problems that they are able to solve, which are not necessarily the problems that need to be solved. Things are harder for practitioners. They have to solve the problems that they are given, which are the problems that have to be solved. If an academic is unable to solve a problem, or comes up with the wrong solution, it can be passed off as a learning experience. It is a lot more serious for a practitioner (Powell, 2013).

1.3. Methodology

The methodology used in this dissertation is based on observations, surveys, shaking table tests, literature study, and computer analysis and design.

1.3.1. Observation of past Earthquake Damages, Surveys, and Shaking Table Test

Almost every year earthquake disasters occurred in many places in Indonesia and caused damage to non-engineered constructions. Through careful observation and study of earthquake damage, the great forces of earthquakes impact to the structure can be recognized.

From observation of structural performance of buildings during an earthquake, the design aspects of buildings, including the qualities of materials, techniques of construction and site selection can be identified and reviewed. The results of those observations have contributed significant information to engineers, architects, building officials, and others engaged in extending the knowledge of earthquake engineering and how to design earthquake resistant buildings that can withstand when shaken by earthquakes. This is particularly true for non-engineered constructions since their earthquake resistant design is mostly based on "observed behavior of such buildings during past earthquakes", and engineering judgment.

In a time span of 40 years, the author surveyed, documented and studied 49 damaging earthquakes in Indonesia and identified a good earthquake resistant feature as well as weaknesses and typical damage of non-engineered construction when shaken by earthquakes. In 1978 (Boen, 1978), the author wrote a simple manual regarding earthquake resistant features for non-engineered construction in Indonesia and after learning from 49 damaging earthquakes, it can be confirmed that almost all of the content of that manual is still valid. Based on the lessons learned, in 2005, the author published a simpler guideline booklet dealing with non-engineered masonry construction only (Boen, 2005). Most of the details in those guidelines shown are based on the prevailing construction practice in Indonesia. However, the detailing shown can be applied in other seismic areas. It is hoped that the guideline is useful for the common people in earthquake prone areas, especially in developing countries and for stakeholders involved in reducing the impact of future earthquakes.

Past earthquakes damage showed that the damage and/or collapse of the non-engineered constructions were caused by the unavailability of standard building materials and poor workmanship, such as incorrect connection detailing, poor quality mortar, poor quality concrete mix. With the extreme pressures of a great demand for new houses together with a limitation of resources available, including finance, skills and building materials, the tendency has been for the standards to fall from those traditionally established. This is evident from surveys and tests in several places in Indonesia that were done by National Graduate Institute for Policy Studies (GRIPS), Japan International Cooperation Agency (JICA) and Universities.

In February 2006 during the reconstruction of houses in Aceh after the December 26, 2004 earthquake and tsunami, GRIPS with financial support of Building Research Institute (BRI), Japan, in cooperation with the Center for Disaster Mitigation, Bandung Institute of Technology set up building materials testing in Aceh. From 28 locations surveyed, 46.43% quality of bricks was below standard class III with minimum compressive strength 60kg/cm^2. From 41 surveyed locations, 58% concrete quality was also found below minimum standard of 125kg/cm^2 (Building Research Institute, 2006).

In June to July 2009, JICA with Polytechnic of Manado conducted survey of local building materials in Manado, Bitung and Tomohon, North Sulawesi (JICA Manado Survey Team, 2009). The surveys were done after the 7.4 SR earthquake in Talaud District, North Sulawesi in 2007. Survey results from 30 brick kilns, 20% quality of bricks was below standard class III. The average brick compressive strength is 77.68 kg/cm^2. Mortar tests were conducted in 4 locations and the result showed that the mortar compressive strength is varied from 54.4-116.3kg/cm^2. For concrete mix 1pc:2sand:3gravel, the average concrete compressive strength is only 94.32 kg/cm^2. This low compressive strength is caused due to bad habits of the workers using excessive water in concrete mix. The workers said that if they used excessive water, the casting process will be easier; they do not need more effort to compact the concrete and get a smooth concrete surface when removing the formwork. Such incorrect understanding is common among construction workers in Indonesia.

From December 2009 to March 2010, JICA in coordination with the Department of Civil Engineering; University of Padang conducted surveys of building materials in Padang, Pariaman, and Padang Pariaman District in West Sumatra (JICA - Jurusan Teknik Sipil Universitas Negeri Padang, 2010). These 3 cities were chosen because the locations are close to the epicenter of the September 30, 2009 earthquake which caused many buildings collapsed. The survey included brick dimension, brick compression test, concrete compression test and plain bar tensile test. From survey in 16 locations, the dimension of the brick generally differs from place to place and from kiln to kiln. Even different batches of the same kiln sometimes do not match so far as their qualities, sizes, and strength are concerned. From survey of brick compressive strength in 45 locations, it was found that the average compressive strength was only 26.8 kg/cm^2. Concrete compressive strength was found below standard. From survey and testing in 29 locations, it was found that the average concrete compressive strength was only 64.54 kg/cm^2. JICA also made a survey for plain reinforcing bars that are available in the market. It was found that all diameters of reinforcing bars are varied and less than the actual size. For example, the diameter for reinforcing bar ∅6mm is only 4.22mm; for ∅8mm: 6.28-7.76mm; for ∅10mm: 7.88-9.82mm, and for ∅12mm: 9.88-11.82mm. The average yield stress of plain bar in all survey areas is 2930 kg/cm^2.

Once again JICA made the third survey in North Sumatra and Padang Pariaman District, West Sumatra from October 2011 to March 2012. The survey was held in 2 cities in North Sumatra (Sibolga and Gunung Sitoli), 6 districts in North Sumatra (Simalungun, Langkat, Tapanuli Tengah District, Nias, Nias Utara, and Nias Barat), and also 1 district in West Sumatra (Padang Pariaman). The survey results again confirmed that the materials quality commonly used to build was below standard and the workmanship quality went down from those traditionally established. In 100 construction location of houses, the average brick compressive strength is 38.9 kg/cm^2; the average concrete compressive strength is 56.4 kg/cm^2; and the average mortar compressive strength is 76.8 kg/cm^2. Almost in all construction sites it was found that the construction workmanship did not follow the principles of earthquake resistant construction. For example, too much water in concrete mix; the stirrups were not bent 135°; no proper detailing in beam-column connections; no anchorage between structural elements, from foundations to tie beams, from walls to columns; the roof trusses did not anchor to ring beams or columns, etc.

Apart from the degrading of the building discipline as evident from the surveys, there is another factor that makes the houses more vulnerable to earthquakes, namely that there is no maintenance culture in the society. Most houses are dilapidated due to lack of maintenance and this also contributes to the damage and collapse of non-engineered constructions.

Because many developing countries and particularly Indonesia did experience many destructive earthquakes, almost every year and caused a lot of casualties and economic loss, many researchers have conducted shaking table tests to study the seismic behavior of vulnerable masonry non-engineered constructions. On December 27, 2007 Mie University in cooperated with NWFP University of Peshawar, Pakistan conducted a one-brick thick wall masonry houses shaking table test in Tsukuba, Japan (Minowa, et al., 2010). Although this test was not exactly following the Indonesian prevailing practice of constructing masonry houses, similar unreinforced one brick thick masonry buildings can be found in Indonesia. 2003 Bam earthquake acceleration record with amplitude 0.815g and 1995 Kobe earthquake acceleration record with amplitude 0.918g were applied for this test. The result showed that the masonry house was rigid and earthquake resistant if constructed with tight supervision.

On July 4, 2008, National research Institute for Earth Science and Disaster Prevention (NIED), and MIE University in cooperation with Building Research Institute, Mitsuishi Fire Brick Co. Ltd and Tokyo Denki University also conducted shaking table test of commonly Indonesian confined masonry house. The test was based on Indonesian prevailing practice to build masonry houses as stipulated in Constructing Seismic Resistant Masonry Houses manual (Boen, 2005). The model was built by unskilled labor and without soaking the bricks first. The quality of bricks and mortars used were also low. 2007 Pisco earthquake record with amplitude 0.33g and 1995 Kobe earthquake record with amplitude 1.299g were applied for this test. The result showed that the non-engineered masonry house collapsed due to poor quality of materials and also poor workmanship (National Research Institute for Earth Science and Disaster Prevention (NIED), 2008; Minowa, et al., 2010).

Another shaking table test based on Indonesian prevailing practice of non-engineered masonry construction was conducted at Ponteficia Universidad Catolica Peru (PUCP) using similar specifications of Indonesian materials (Minowa, et al., 2010). 2007 Pisco earthquake record, 1995 Kobe earthquake record, and 1970 Peru earthquake record were applied for this test. In this experiment, there are 3 variant models used. The first is masonry walls which are not using reinforced concrete lintel beams over openings and no anchors between walls and columns. The result showed that that model suffered extensive cracks and finally collapsed. The results coincide with past earthquake damages observed. The second model is a house with continuous reinforced concrete lintel beam over the door and windows openings, and also steel anchors between walls and columns at three positions. This model has the same concept with the Indonesian earthquake resistant masonry houses. The result showed that the house survived although cracks occurred in the openings. This proves that if the buildings constructed followed the earthquake resistant principles and with good workmanship, even though cracks occurred when shaken by earthquake, the buildings still can withstand and do not collapse and not endanger human life. The third model had an external wire mesh covering the surface of the walls. The wire mesh was only wrapped to the walls and did not act as ferrocement but more as a safety net. The result showed that wire mesh is a good feature to use as strengthening material of masonry walls. Even if the wire mesh is not bonded to the walls, there is a significant improvement of masonry walls strength.

In order to save lives during a large earthquake, Indonesia with millions of vulnerable non-engineered constructions and the recurrence of earthquakes almost every year, have to start implementing techniques or methods how to reduce the impact of earthquakes. As mentioned before, since rebuilding all existing structures that are not earthquake resistant is costly, the ideal solution would be to discover some techniques or methods to strengthen these non-engineered constructions and improve the local building materials. The method shall be simple, replicable, and affordable; the technology that is feasible to adopt quickly (IAEE, 1980).

In 2007, the author retrofitted a school building in Soreang, Bandung, using sandwich-type construction where the brick-wall acts as a core and ferrocement on both sides of the wall act as skin facings. After a destructive earthquake hit West Sumatra on September 30, 2009, the author used the same retrofitting method for masonry walls and applied in several houses, school buildings as well as engineered buildings. To verify the strength of retrofitting walls with such method, a shaking table test was conducted as a collaborative research between National research Institute for Earth science and Disaster prevention (NIED) and Mie University (Imai & Nakatani, 2012; Hanazato, 2013). Although the test was conducted in Japan, the materials used for this experiment are imported from West Sumatra, Indonesia. The result showed that there is a significant effect of reinforcement using wire mesh. The reinforcement using wire mesh was effective in preventing from collapse of walls. Major cracks in in-plane direction were initiated from the corner of the opening; coincide with past earthquake damages observed.

Although the shaking table test has proved that this retrofitting method is effective to strengthen non-engineered masonry buildings, the method still should be studied theoretically. Considering the vast number of books, papers, and seminar proceedings on the topic of ferrocement and sandwich construction, following is the brief review of the literatures.

1.3.2. Literature Review

1.3.2.1. What is Retrofitting

Masonry buildings, especially in developing countries, have a large portion of the buildings around the world. Most of them are residential buildings and schools, and are occupied by many people and children. The experience of past earthquakes has shown that a great number of masonry structures are vulnerable to seismic actions so that moderate to strong earthquakes can devastate them resulting in a large number of victims and extensive losses (Reinhorn, et al., 1985; Taghdi, et al., 2000; ElGawady, et al., 2004; Ghiassi, et al., 2008; Gesualdo & Monaco, 2011; Ashraf, et al., 2011). This vulnerability is mostly because of several reasons:

- Structures were constructed in a time that there was not any seismic code available. Many older masonry structures currently in use were in fact designed and constructed with little or no consideration of seismic requirements.
- Structures were designed and constructed without following the available seismic code.
- Structures were designed and constructed according to the seismic code, but because of the complexity and lack of information on the behavior of the masonry structures, the code's regulations were not accurate enough.

Seismic vulnerability of masonry structures depends on the configuration and mechanical properties of masonry (Ghiassi, et al., 2008). The mechanical properties of masonry including shear modulus, stiffness, the orientation of the bed joints and the stress state of the joints depend on various factors (Bosiljkov, et al., 2005).

The understanding about the behavior of masonry declined in the first decade of the last century and therefore, the available methods for assessing masonry are not reliable (Gesualdo & Monaco, 2011). The capability of unreinforced masonry walls to resist lateral loads is limited by the strength of both masonry units and bed joint mortar. For in plane loading of unreinforced masonry walls with low axial loads, the failure mode is sliding along bed joints and crack at the bottom corner of the wall (overturning) due to rocking (Taghdi, et al., 2000).

Past earthquakes have shown that most masonry structures are vulnerable, particularly older unreinforced masonry walls, and have a potential for a great loss of life. Therefore, masonry structure has become the subject of a wide range research and retrofitting strategies for masonry structures are in great demand in the last few years. Moreover, based on modern design codes most of the existing URM buildings need to be retrofitted (ElGawady, et al., 2004; Gesualdo & Monaco, 2011). Improving existing retrofitting methods and developing better ones for existing buildings is urgently needed. The decision whether strengthening or retrofitting should be used depends on the seismic resistance of the masonry building and the expected level of damage. In seismic areas, the basic criterion for repair and strengthening is based on the correlation of the expected seismic loads with the resistance of the structural system, i.e. on seismic resistance verification (Churilov, 2012).

Numerous techniques are available to increase the strength and/or ductility of URM walls, both in-plane and out-of-plane direction (ElGawady, et al., 2004). Different strengthening techniques have been developed for masonry buildings; some of them based only on the analysis of earthquake damage and engineering judgment, and have never been actually verified, and some of them based both on earthquake damage observation and experimental investigations, verified in laboratory or by a real earthquake (Churilov, 2012).

In contrast to the more or less limited interventions in stone masonry structures to injection of grout into void parts of the walls, there are various possibilities available for strengthening of brick and block masonry walls. Some conventional methods are surface treatment (ferrocement, FRP layer, shotcrete layer), grout and epoxy injection, external reinforcement, confining masonry walls, constructing new internal or external shear walls or steel braced frames, and prestressing of the walls in vertical and horizontal direction (Taghdi, et al., 2000; Ghiassi, et al., 2008; Ashraf, et al., 2011; Plesu, et al., 2011; Churilov, 2012).

While the above retrofitting techniques are effective, they require a great deal of preparation work, their construction may disturb the ongoing building functions, and the new structural elements may affect the architectural aesthetics of the building (Taghdi, et al., 2000).

In the context of various developing countries where many old structures require retrofitting or strengthening work to mitigate earthquake hazards, a technique which is easy in application, rapid in construction and very low in cost, with no heavy machinery and high-level skilled workers is the retrofitting of damaged masonry-infilled reinforced concrete (RC) frames using ferrocement overlays, and the strengthening of existing infilled reinforced concrete frames with ferrocement (Ghiassi, et al., 2008; Alam, et al., 2009; Ashraf, et al., 2011). In this technique steel welded wire mesh (or hardware cloth) is connected to the surface of masonry through bolts/screws subsequently covered with plaster coating.

Ferrocement overlay has a considerable use in retrofitting unreinforced masonry walls that need to be improved in-plane strength, out-of-plane strength, and ductility (Reinhorn, et al., 1985). Some proposed strengthening of wall corners using ferrocement (Arya, 2007). Retrofitting unreinforced masonry wall using ferrocement is a common technique, but there is not any design guideline for that (Ghiassi, et al., 2008). No reliable mathematical or computational tool is accessible in the open literature to estimate the effect of such a retrofitting technique quantitatively because of the lack of experimental and analytical information on this method, therefore, the rehabilitation procedures are being done based on empirical judgments (Ghiassi, et al., 2008; Alam, et al., 2009).

A numerical investigation using finite element technique of the retrofitting effect of masonry-infilled reinforced concrete frames using ferrocement has been studied by (Alam, et al., 2009). Although the model accounts for material nonlinearities of both concrete and masonry, and the yielding of reinforcing steel, it was only for a single wall with in-plane loading and not represents the behavior of 3D structure.

According to (Shahzada, et al., 2012), application of wire mesh increases the lateral strength capacity of unreinforced masonry walls significantly. A study was carried out to strengthen the existing unreinforced brick masonry walls with ferrocement technique with a potential of constructing new structures with ferrocement.

1.3.2.2. __What is Ferrocement__

Ferrocement is a thin composite consisting of cement mortar matrix reinforced with a small diameter wire mesh encapsulated in the matrix (Center for Building Technology, 1974; Sakthivel & Jagannathan, 2011). The thickness of ferrocement is approximately 10-50 mm and uses rich cement mortar; no coarse aggregate is used; and the reinforcement consists of one or more layers of continuous / small diameter steel wire / weld mesh netting. Excessive mortar thickness can lead to premature cracks.

Ferrocement with more than one layer of steel mesh can be considered as a composite consisting of several lamina layers stacked on top of each other (Figure 2). A lamina consists of a composite of one single layer of steel mesh embedded in two layers of cement mortar.

Ferrocement and reinforced concrete are similar, made of structural concrete materials or cement-based composites. (ACI Committee 549, 1999; Naaman, 2000; Sasiekalaa & Malathy, 2012). The engineering properties of ferrocement structure are similar to reinforced concrete, and in some applications it performs better. Reinforced concrete and ferrocement use similar matrix and reinforcement materials; the model, analysis, and design of ferrocement follow the reinforced concrete principles, techniques, and philosophy.

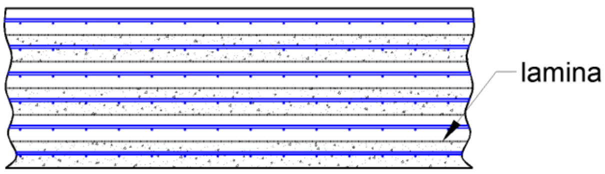


Figure 2 – Ferrocement as a Laminated Composite

Steel wire mesh for ferrocement are square woven or welded meshes, chicken hexagonal shape wire mesh, or expanded metal sheet (usually used when plastering) (Figure 3). If the steel mesh consists of a square mesh, the mechanical properties in two principle directions are and can be assumed isotropic in the two principle directions.

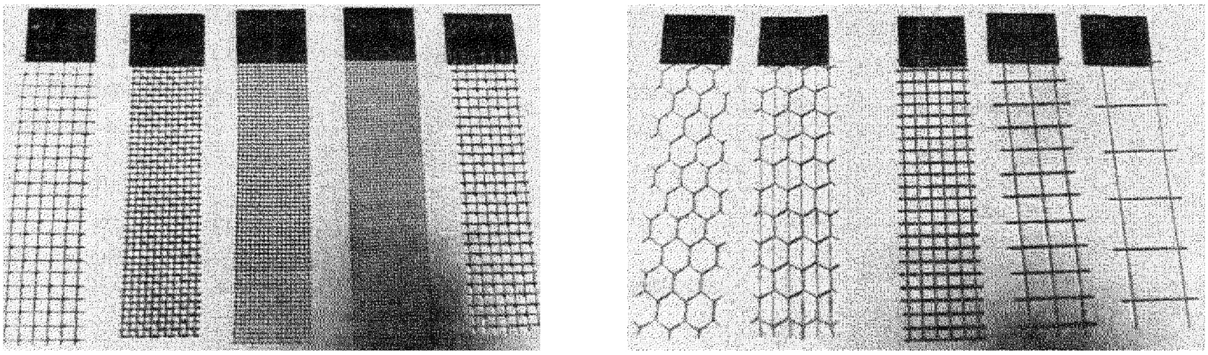


Figure 3 – Types of Wire Mesh for Ferrocement (ACI Committee 549, 1999)

One of the advantages of ferrocement is the fact that it does not disintegrate after failure unlike masonry walls, therefore, this can reduce falling hazard during earthquake. As it is known thin unreinforced masonry walls failed suddenly and cause brittle failure. On the contrary ferrocement walls crack at lower loads, but it needs greater loads to widen the cracks that can lead to failure.

Generally, the main characteristics of ferrocement are: (Naaman, 2000; Sakthivel & Jagannathan, 2011; Blogger, 2012; Sasiekalaa & Malathy, 2012; Kulkarni & Gaidhankar, 2013; Ferrocements Pvt. Ltd., 2013)

- Homogenous-isotropic material with up to 40% of yield in two directions.
- High tensile strength and high modulus of rupture.
- Better impact and punching shear resistance than reinforced concrete.
- Light-weight than reinforced concrete.
- High level of impact and cracking resistance, toughness, and ductility.
- Highly waterproof, energy absorbing, and resilient than reinforced concrete.
- Have high deformation before collapsed.
- Good leakage characteristics and durable under various exposures.

The reinforcement parameters in ferrocement are represented by volume fraction of reinforcement, V_r, specific surface of reinforcement, S_r, and the elasticity modulus of reinforcement, E_r (ACI Committee 549, 1999; Naaman, 2000; Bangladesh National Building Code, 2012).

- Volume fraction of reinforcement, V_r, is the total volume of reinforcement divided by the volume of composite (reinforcement and matrix). If reinforcement placed in the middle, it must be considered in evaluating the resistance of tensile members.
- The specific surface of reinforcement, S_r, is the total bonded area of reinforcement (interface area or area of the steel that comes in contact with the mortar) divided by the volume of composite.
- The effective modulus of the reinforcing system E_r may be different because the elastic modulus of a mesh (steel or other) is not necessarily the same as the elastic modulus of the filament (wire or other) from which it is made. For welded square mesh, E_r may be taken equal to the elastic modulus of the steel wires; the typical value of E_r is approximately 2,000,000kg/cm^2. For other meshes, E_r may be determined from tensile tests on the ferrocement composite where the mesh is embedded in a matrix. In mesh systems with very large deformations under load, the effective modulus will be smaller and crack widths larger, subsequently the mortar cover will spall.

Like reinforced concrete, the mechanical properties of ferrocement element under compression mainly depend on the mix design properties. The mechanical properties of ferrocement including tension, bending, and cracking can be found in ACI-549R (ACI Committee 549, 1999). The properties of ferrocement range from plain cementitious matrix to the properties of a composite containing a large volume fraction of uniformly distributed reinforcement leading to a high specific surface of reinforcement.

There are some distinctive behavior of ferrocement in tension compared to reinforced concrete, namely the cracking behavior, the maximum elongation at failure, the stress at first cracking, and the modulus of the mesh system.

Similar to concrete, the tensile performance of ferrocement can be grouped into three, namely, the pre-cracking phase, post-cracking phase, and finally the post-yielding phase (Naaman, 2000; Blogger, 2012). At pre-cracking phase, reinforced concrete as well as ferrocement member subjected to upwards tensile stress behaves linear elastic until the first crack appears. The crack width of first cracking in ferrocement is ranging from 0.005mm, which needs a microscope to see, to 0.03-0.1mm, which can be visible by naked eye. The significant difference between concrete and ferrocement lies in post-cracking phase, where cracking starts and during which crack formation stabilizes. The ferrocement member will enter the multiple cracking and it can be extensive, eventually continuing to a point when the number of cracks stabilizes and the mesh starts to experience yielding. With proper reinforcement, the multiple cracking of matrix will be fine and similar to the ductile characteristics of the reinforcement. The number of cracks will continue to grow with the increase in the tensile force or stress. A true structural crack is formed when a group of micro-cracks, which develop as soon as the elements are subjected to tensile loading, link together and separate the element into two totally unconnected parts. Stabilization stress in ferrocement is reached when there are no more new cracks and the behavior of ferrocement is controlled by reinforcement.

The specific surface of reinforcement has been found to influence the first crack in tension, as well as the width of the cracks. The stress at first cracking increases linearly with the specific surface. For bending, the rate of increase specific surface of reinforcement is not as significant as in tension.

The tensile strength of ferrocement is directly proportional to the specific surface of reinforcement in the loading direction (Naaman, 2000; Sasiekalaa & Malathy, 2012). However, the elongation at failure also increases when the volume fraction of reinforcement (the number of layers of reinforcing mesh) increases.

The tensile strength of ferrocement depends on the tensile strength of its reinforcement (the yield strength of steel meshes), the mesh orientation, and the direction of applied loading. The tensile strength of ferrocement can be estimated based on the yield strength of the reinforcing mesh. If the crack width and spacing remain within limits, increasing the tensile strength of the mesh reinforcement leads to direct increase in composite tensile strength.

The elasticity modulus is a fundamental property of material, defined as a measure of the change in stress due to a change in strain between two points of the tensile stress-strain response of a material. Generally the tangent or initial modulus of ferrocement is implied when the composite still behave linear elastic. After the first crack occurs, the effective modulus of ferrocement composite depends on its volume fraction of reinforcement and the extent of multiple cracking.

The bending resistance of ferrocement increases with the volume fraction of reinforcement. Actually, bending represents the influence of tensile and compression properties, which are controlled by mortar compressive strength, mesh type, mesh properties and mesh orientation. If bending in ferrocement is considered in one direction only, the two-way nature of the mesh reinforcement will give some additional strength and safety. The average crack width in ferrocement bending elements is primarily a function of the tensile strain in the extreme layer of mesh and the transverse wire spacing.

1.3.2.3. *What is a Sandwich Structure*

Sandwich-type construction is a composite construction consisting of three integrally attached layers so that the material properties of each one can be utilized for the structural advantage of the whole assembly (Allen, 1969; Baker, et al., 1972; Mukundan, 2003; Kormanikova, 2003; Ratwani, 2004). The middle layer of the sandwich is the core; the outer two layers are the skin facing sheets.

All the references deal with basic concepts of sandwich-type construction and development of equations for the deflection of a sandwich with similar thin, flat-faces. The major difference, compared to conventional stress analysis, is the need to account for shear deformation and creep of the core. In normal structures, shear deformation is so slight that it can be neglected, and the properties of the materials are little affected by sustained loads.

In modern construction, sandwich structures are used for high-technology application such as aircrafts, spacecraft, satellites or F-1 racing cars, where weight savings in structural elements are the key to dramatic technology. For high-tech application, these structures should be as light as possible while having high stiffness with sufficient strength. Sandwich structures have also been widely used in sandwich panels; these kinds of panels can be in different types such as FRP sandwich panel, aluminum composite panel etc.

Generally, the skin facing sheets are very thin relative to the overall thickness of the sandwich and the elastic modulus of the facing-sheet material is much larger than the corresponding effective modulus of the core. It is a type of stressed-skin facings construction in which the facings resist nearly all the applied edgewise (in-plane) loads and flatwise bending moments. The thin facings provide nearly all the bending rigidity to the construction. The core spaces the facings and transmits shear between them so that they are effective about a common neutral axis. The core also provides most of the shear rigidity of the sandwich construction. By proper choice of materials for facings and core, the bending stiffness and stiffness to weight ratio of the sandwich is greater than a single solid plate of same total weight and same material as that of the faces. Because of that, sandwich construction results in lower lateral deformations, higher buckling resistance and higher natural frequencies than do other constructions.

In general, there are three types of cores commercially available. These are cellular cores, corrugated cores, and solid cores.

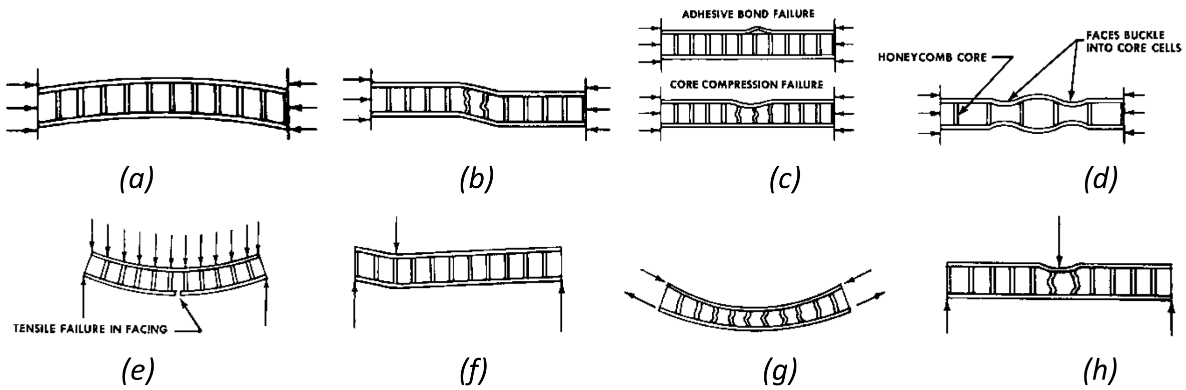
Usually, balsa wood and plywood are used as solid cores and in such cases the core itself can be used as a structural member. Balsa wood and plywood are strong in three planes for bending and shear rigidities. Because of that, those properties, the strength of the resulting sandwich structures will be greater even though using the same skin facing sheets if compared to sandwich structures using cellular or corrugated cores.

Cores have basic mechanical and physical properties and in designing sandwich structures, those properties must be analyzed. Some of these properties are included in the basic analytical parameters of sandwich structures. Those properties dictate the behavior of the sandwich structures for its stability, stress, and deflection modes.

There is a basic difference in orthotropic or isotropic shells if compared to sandwich shells in modern construction, usually in sandwich construction the transverse shear stiffness must be included in the analysis because sandwich structures has a low transverse shear stiffness.

In high-tech sandwich structures, cores are usually made of metallic or composite material corrugations. Subsequently, the corrugated core maybe welds bonded, glue bonded, or riveted bonded to the metallic face sheets. In case of composite face sheets, the core can be bonded to the face sheets. A sandwich construction has the following advantages:

- High ratio of bending stiffness to weight as compared to monolithic construction.
- High resistance to mechanical and sonic fatigue.
- Good damping characteristic.
- Improved thermal insulation.
- No mechanical fasteners, hence, no crack initiation sites.



(a) General Bucking Panel; (b) Shear Crimping; (c) Face Wrinkling; (d) Intracell Buckling; (e) Tensile Failure in Facing; (f) Transverse Shear Failure; (g) Flexural Crushing of Core; (h) Local Crushing of Core

Figure 4 – Failure Modes in Sandwich Structures (Ratwani, 2004)

Failure modes in high-tech sandwich structures with cellular cores or corrugated cores are different from those in monolithic structures. The general failure modes of high-tech sandwich structures are shown in Figure 4.

Generally, high-tech sandwich construction should be analyzed for three modes of failure:

- Material failure, meaning the applied load causes the material stresses to exceed the permissible stresses
- General-instability failures, meaning both the facings as well as the core fail.
- If the core of the sandwich is constructed cellular materials, it is possible that the intra-cell buckling can occur. Also, when the facing sheets of the sandwich elements

is subjected to axial compression, face sheet wrinkling may occur. Those two failures are local-instability failure.

Although a considerable amount of theoretical information is available concerning the general instability of sandwich shells, not enough test data are available to obtain design curves directly. Therefore, the design curves for homogenous isotropic shells are used to reduce the theoretical buckling loads for sandwich shells to design-allowable buckling loads.

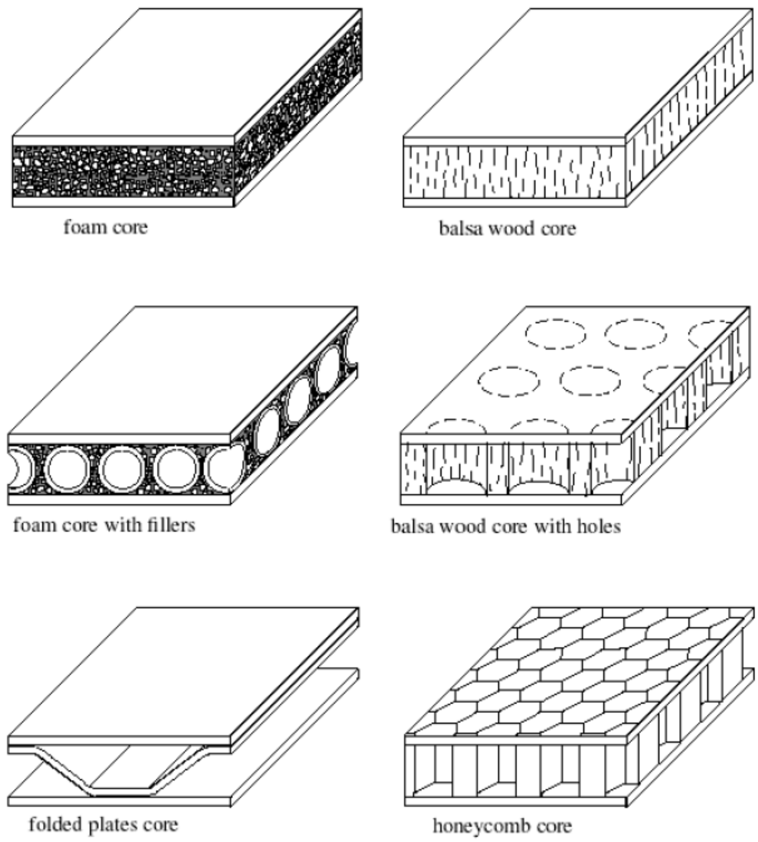


Figure 5 – Traditional Sandwich Panels (Kormanikova, 2003)

1.3.2.4. Retrofitting Non-Engineered Construction not Considered in Disaster Risk Reduction in Indonesia

Resources allocated for disaster risk reduction (DRR) for non-engineered construction is very small and the decision for the allocation is not clear nor is it based on a systematic approach. The scope of DRR investment also remains limited, focusing, in particular, on: public awareness, disaster education and training and institutional and legislative frameworks. Other DRR related measures such as risk assessment and research are still not the main aim and resources were not allocated for among others strengthening of buildings prior to the 2009 West Java and West Sumatra earthquakes.

Community based DRR investments tend to provide positive results. This was illustrated by the community's response that they were not panicked and were more prepared when the earthquakes struck. However, the devastating effect of the earthquakes still destroyed buildings and caused a lot of people to lose their lives.

The fact that risk assessment has been conducted by several institutions shows that DRR actors have become more systematic in the implementation of their DRR programs. Many of the risk assessments were carried out by NGOs but are not connected with the risk assessment conducted by specific agencies (BPPT, BMKG, Bakosurtanal). There was no comprehensive provincial and districts risk information that allows more coordinated and comprehensive risk reductions. At village level, risk assessment was limited in scope and mostly followed the predetermined objectives of a particular project, e.g. preparedness.

Lack of awareness and understanding about the purpose and meaning of DRR among decision makers in Indonesia is the main cause of small resources allocated for DRR. There is little initiative to make this audience the target for training and awareness raising about DRR.

There are some initiatives to use risk assessment as the basis for risk reduction. However, risk assessment and analysis has not yet been maintained and managed as a mechanism or system. Risk assessment also tends to be viewed and practiced as a means to produce a specific product (e.g. map), instead of a process to build consensus and common understanding in particular areas.

Resources allocated for DRR related activities are relatively limited, the highest investment in public awareness, disaster education and trainings were considered the highest priorities in both provinces (48% for West Java and 63% for West Sumatra). Public awareness, education and training also received high attention, particularly from the government and NGOs, however, the coverage was relatively limited to some schools (normally formal education) and beneficiaries, compared to the size of the populations. Activities also tend to be carried out on preparedness related knowledge and skills **and less on structural mitigation, which aims to strengthen the buildings** and infrastructure where public or children normally gather. Only recently has the national government launched a sizeable school safety campaigns.

With regards to research, only limited activities were carried out by the government and NGOs in cooperation with universities. Universities were only acting as consultants in the researches. Government resources for research are limited.

Community-Based projects have had positive impacts on the communities. Communities have become more aware of the potential risks that may happen. However the impact of community based approaches seems to be limited in scope. Community based initiatives tend to stay within the targeted communities rather than to influence larger communities. Community based approaches with limited application were introduced, however the impact of community based approaches seemed to be limited in scope, therefore, community based approaches must still be promoted but at the same time more work needs to be done to find out how (Sagala, 2010).

According to Director for Special Area and Disadvantaged Region National Development Planning Agency (BAPPENAS), there are several issues on disaster management in Indonesia (Hadi, 2008):

1. Lack of management capacity on disaster response:
    a. Delay in the management of emergency response
    b. Lack of coordination in planning and programming for post-disaster recovery

   c. **Institutional framework is more focused on emergency response, rather than post disaster recovery**
   d. **Funding more emphasizes on emergency response**
2. Lack of understanding in disaster risk reduction:
   a. Lack of understanding in the preparation of disaster preparedness and risk reduction
   b. Lack of institutional performance in the management of risk reduction
   c. **Lack of planning and programming for risk reduction**
   d. Lack of incorporating disaster risk reduction into spatial plans

Retrofitting of non-engineered construction to better withstand earthquakes should be viewed as part of a comprehensive response to the overall housing problem in Indonesia. The need and urgency for such retrofitting is made clear by the country's long history of earthquakes, in conjunction with current problems in the housing sector.

It is clear that Indonesia cannot afford the additional burden of replacing houses lost in a disaster. This highlights the need to retrofit the existing housing stock to preclude the need to replace them after a disaster. In a previous study of houses of non-engineered construction in disaster-prone areas the following observations were made.

Structural retrofitting of non-engineered construction can be viewed in terms of its advantages with reference to disasters as well as its contribution to the resolution of existing housing problems. In terms of disasters, by emphasizing modification and retrofitting, the number of units lost to a disaster will be lowered and the reconstruction burden on both the government and the people will be reduced. A house that withstands a disaster not only represents a safe refugee for its occupants, but also eliminates the tremendous discontinuity and economic burden resulting from damage to the building and it represents a lessening of the foreign exchange problem and reduction of further strains on a reconstruction economy.

Retrofitting is cheaper than replacement of substandard units, and many of the measures taken to improve disaster resistance will help make the housing more livable as well as more durable. Furthermore, retrofitting places the majority of the burden on the homeowner rather than on the government; thereby enabling policy-makers to spread financial resources to a greater number of beneficiaries (AusAID, 2013)**.**

From those references, it is clear that disaster risk reduction in Indonesia does not consider retrofitting of non-engineered construction as an important measure.

1.3.3. Computer Analysis and Design

The availability of hardware and software such as SAP2000 and ETABS greatly simplify the tasks of experts to analyze structures subjected to earthquakes shakings and make it more quickly and efficiently. Non-engineered buildings can be engineered using advanced technology contained in SAP2000 and ETABS. By studying the damage of buildings after earthquakes and subsequently study the analysis results using the advanced computer technology equipped with a graphical display, the experts can identify the weakness of structures. These weaknesses can be corrected and subsequently used to build safer, thus using technology for building safer houses.

In Indonesia, since 2001, the author introduced "engineering non-engineered construction" and used softwares to model, analyze, and design non-engineered buildings in Bengkulu, Aceh, Yogya, and West-Sumatra. The purpose of analysis is not to simulate the actual behavior, but to get reliable information that there is a correlation between the observed damages and the results of the analysis. The correlation is not perfect, but is good enough to get a good idea to build appropriate non-engineered constructions that can withstand earthquakes.

1.4. Structure of Dissertation

The content of this dissertation is divided into six chapters. **Chapter 1** gives an overview of the dissertation. The background, objective, and methodology are presented briefly.

In **Chapter 2** “the earthquake problems in Indonesia“ will be elaborated since in fact, considerable research and available guidelines regarding non-engineered construction were available since almost 35 years ago, however, until today, Indonesia is still experiencing damages of non-engineered construction that caused large number casualties and economic loss during destructive earthquakes. The problems should be reviewed so that the necessary actions can be taken to prevent and/or reduce damage and casualties caused by the next earthquakes.

There are two major issues related to non-engineered construction in Indonesia. The first is related to the unsafe non-engineered construction stock in Indonesia; and the second is related to ineffectiveness of disaster risk reduction. These two main issues will be discussed in order to get an overall view of the problems. **Chapter 2** will be closed with alternative solution to prevent further damage and/or collapse of unsafe non-engineered constructions technically.

Chapter 3 describes the definition, the history and types of non-engineered construction in Indonesia. From this chapter, the dissertation will be focused on masonry construction; the type of non-engineered constructions which suffered most damage and collapse due to earthquakes. A good earthquake resistant design feature will be described, namely half-brick-thick masonry buildings with reinforced concrete framing, consisting of the so called “practical columns and beams”. The characteristics of masonry as a composite structure made of masonry units and mortars and some issues that should be considered to build earthquake resistant masonry buildings are also elaborated briefly. The sequence how to construct seismic resistant masonry houses will be shown using graphic illustration with pictures in the actual construction.

Some experiments of non-engineered masonry construction that were conducted will be presented as a complement to show that if such type of construction actually is earthquake resistant if they are built properly using good quality materials, good workmanship and all building components (foundation, columns, beams, walls, roof trusses, roofing) are tied each other, so that when shaken by earthquakes, the building will act as one integral unit.

Chapter 3 also describes the general causes of damage and collapse of masonry buildings during earthquakes; mostly due to poor quality of materials and poor workmanship. Past testing and surveys related to non-engineered construction in Indonesia conducted by foreign government agencies and universities will be summarized. At the end, issues related to non-engineered construction in Indonesia will serve as a closing topic for this chapter.

Continue to **Chapter 4**, design basis of non-engineered constructions will be described. Observations from survey of past earthquake damages of non-engineered constructions serve as a basis how to design non-engineered constructions. The investigation of past earthquakes and their effects on various types of structures have contributed significant information. The causes and typical damages are explained and analysis of the mechanism of damage is performed.

The principle loading that causes damages are identified and also elaborated in Chapter 4. Failure mechanism of non-engineered masonry buildings due to seismic shaking is mainly caused by out-of-plane bending failure of walls, and / or in-plane shear failure. For unreinforced masonry buildings, the out of plane loading plays a significant role as a cause of damage and/or collapse of walls due to earthquake.

With the vast development of computing technology and the availability of softwares nowadays, mechanism of damage can be confirmed by analysis using computer models. Example of non-engineered constructions analysis and the correlation between the observed damages and the results of the analysis will be presented at the end of **Chapter 4** to give a good idea how to build appropriate non-engineered constructions that can withstand earthquakes.

One of method to avoid collapse of these unsafe non-engineered constructions is by retrofitting masonry construction using ferrocement. The proposed retrofitting method will be described in **Chapter 5**. This retrofitting method is simple, affordable and replicable for non-engineered constructions in Indonesia to reduce future casualties. The step by step procedures of retrofitting using ferrocement will be elaborated. Application of sandwich theory for the proposed retrofitting method will be described briefly; the brick-wall acts as the core and ferrocement on both sides of the wall act as skin facings.

Example of analysis and design utilizing existing commercial software also will be shown in **Chapter 5**, followed by experiment that was conducted in Japan related to the proposed retrofitted method as a proof that this proposed method is reliable to strengthen unsafe masonry non-engineered constructions.

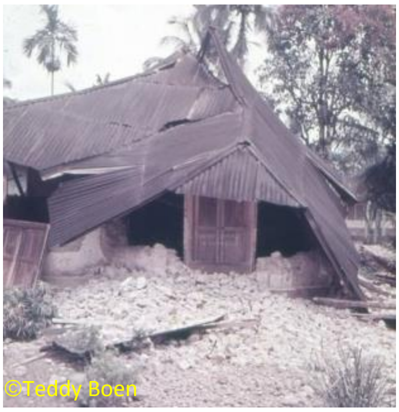
Finally, **Chapter 6** serves as the closing chapter, explaining briefly that the proposed retrofitting method can be applied in other developing countries with similar unreinforced masonry. **Chapter 6** also outlines some thoughts for the way forward to improve disaster risk reduction in Indonesia, among others regarding disaster risk reduction training program that should be effective using a multi-sectoral approach; training for disaster risk reduction for each target groups (policy makers, national planners, project staff, community groups, NGOs, and trainers themselves) which have different training needs is explained briefly.

Chapter 2 Indonesian Earthquake Problems

Almost every year earthquake disasters occur in many places in different parts of Indonesia and cause damage and destruction of buildings. Most of the damaged and/or collapsed buildings are people's houses so called as non-engineered constructions (Figure 6). Despite of the many human casualties and the severe impact on the regional economy and development, it seems that relatively little is being done to prepare, prevent or mitigate the effects of future earthquakes. The earthquakes are repetitions of all past occurrences and are a demonstration that not much has been done with regard to non-engineered constructions. This is clearly demonstrated by the July 2, 2013 earthquake in Aceh Tengah and Bener Meriah in Aceh Province, where many non-engineered houses are heavily damaged and collapsed, taking 36 lifes.

Sumbawa – 1954

Bali – 1976



Pasaman – 1977

Mataram – 1979

Figure 6 – Damage of Non-Engineered Constructions due to Earthquakes in Indonesia

Tasik / Garut – 1979

Sukabumi – 1982

Flores – 1982

Tarutung – 1987

Majalengka – 1990

Flores – 1992

Halmahera – 1994

Liwa – 1994

Figure 6 (cont'd) – Damage of Non-Engineered Constructions due to Earthquakes in Indonesia

Palu – 1995

Kerinci – 1995

Biak – 1996

Pinrang – 1997

Pandeglang – 2000

Bengkulu – 2000

Karangasem – 2004

Padang Panjang – 2004

Figure 6 (cont'd) – Damage of Non-Engineered Constructions due to Earthquakes in Indonesia

Nabire – Feb 2004

Alor – 2004

Nabire – Nov 2004

Palu – 2005

Nias – 2005

Yogyakarta – 2006

West Sumatra – 2007

Bengkulu – 2007

Figure 6 (cont'd) – Damage of Non-Engineered Constructions due to Earthquakes in Indonesia

Dompu – 2007

Simeulue – 2008

Manokwari – 2009

West Java – 2009

West Sumatra – 2009

Aceh Tengah – 2013

Figure 6 (cont'd) – Damage of Non-Engineered Constructions due to Earthquakes in Indonesia

One- and / or two-story school buildings in Indonesia can be considered as non-engineered constructions and also, like other non-engineered constructions are often damaged and destroyed when shaken by earthquakes (Figure 7). They are even more vulnerable due to larger spacing between walls in both directions if compared to residential buildings. Failure of schools may result in cutting short the lives of the future intelligentsia in a country and cause terrible upsetting to the parents of the children who lose their lives. Needless to say that collapsed and or damaged school buildings will disrupt the education activities.

Besides the function of education, school buildings in rural and semi urban areas of developing countries are used as multi-purpose community buildings, especially for providing shelter to population in times of distress. This happens because the school building may be the only public building in a village or group of villages. This calls for greater safety as well as durability in the construction of educational buildings (School of Research and Training in Earthquake Engineering University of Roorkee, 1977).

Bali – 1976

Aceh – 1983

Sukabumi – 1982

Tarutung – 1987

Flores – 1992

Halmahera – 1994

Liwa – 1994

Kerinci – 1995

Figure 7 – School Buildings Damage due to Earthquakes in Indonesia

Biak – 1996

Pandegelang – 2000

Bengkulu – 2000

Karangasem – 2004

Padang Panjang – 2004

Nabire – Feb 2004

Alor – 2004

Nabire – Nov 2004

Figure 7 (cont'd) – School Buildings Damage due to Earthquakes in Indonesia

Simeulue – 2005

Nias – 2005

Yogyakarta – 2006

West Sumatra – 2007

Bengkulu – 2007

Dompu – 2007

Simeulue – 2008

West Java – 2009

Figure 7 (cont'd) – School Buildings Damage due to Earthquakes in Indonesia

West Sumatra – 2009

Aceh Tengah – 2013

Figure 7 (cont'd) – School Buildings Damage due to Earthquakes in Indonesia

With the re-occurrence of the same mistakes until today, "the earthquake problem in Indonesia" should be reviewed so that the necessary actions can be taken to prevent damage and casualties caused by the next earthquakes.

There are two major issues related to non-engineered construction in Indonesia as described in Figure 8. The first is the unsafe non-engineered construction stock in Indonesia and the second is the obstacles of disaster risk reduction.

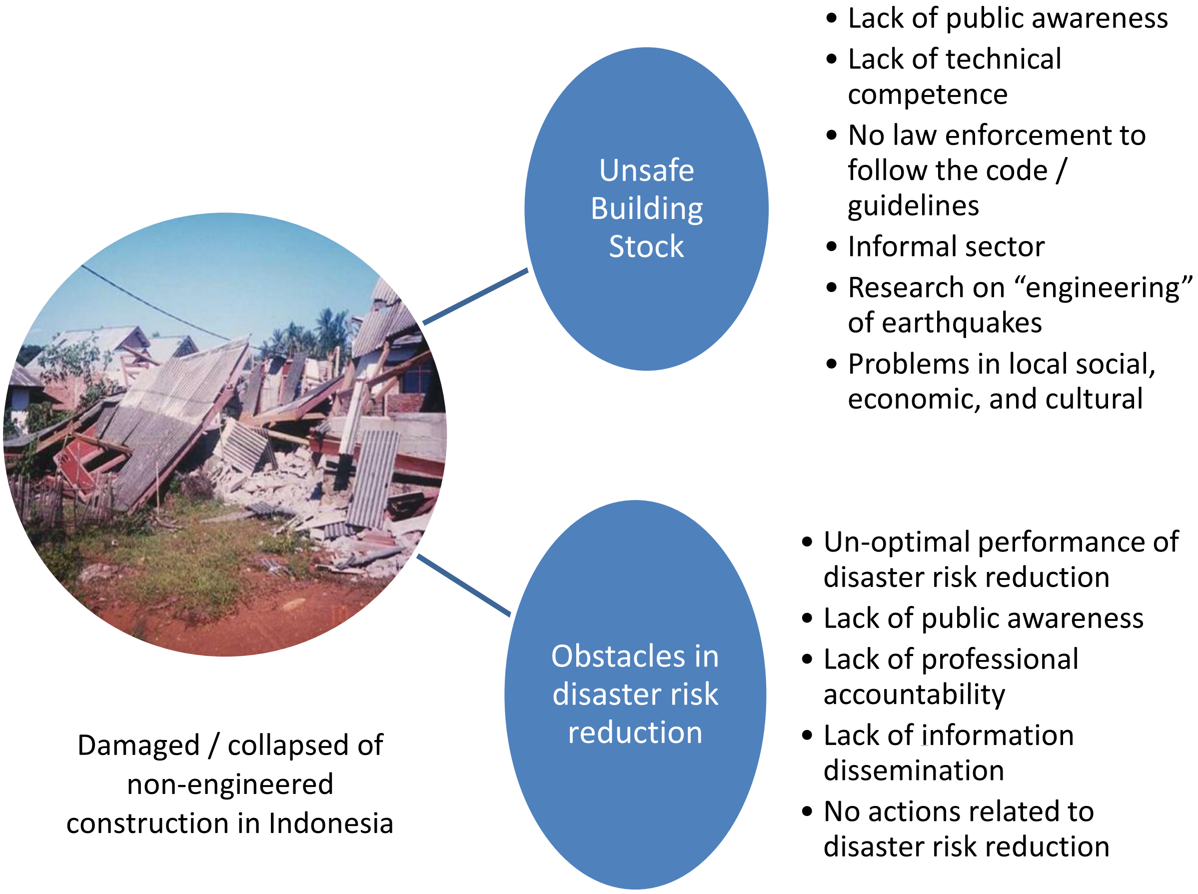


Figure 8 – Issues related to Non-Engineered Construction in Indonesia

2.1. Unsafe Non-Engineered Constructions in Indonesia

Every time there is a damaging earthquake in Indonesia, all printed as well as electronic media provides a wide coverage about the earthquake related problems. Various government agencies / ministries announced plans to take care of the problem. Many experts and scientists are being interviewed by newspapers, tabloids, magazines as well as TV stations and issue numerous opinions regarding what has happened and offer solutions to prevent similar happenings in the future. Needless to say, all “experts” that were interviewed considered their field of expertise as the most important and therefore, the media is filled with all sorts of opinions which are confusing the common people. However, the actual real problem is the unsafe of non-engineered constructions so that it can be damaged and/or collapsed due to earthquakes (Figure 6). Very few experts are highlighting the need to make all non-engineered constructions earthquake resistant (Boen, 2008).

Subsequently many seminars, workshops, trainings related to earthquakes are held. The community is lead to believe that their earthquake safety is taken care of, until the next earthquake shows that not much has been done since the last damaging earthquake. With the damages and casualties that occurred, particularly in the past seven years, namely in Yogyakarta (May 27, 2006), West Sumatra (March 6, 2007), Bengkulu (September 12, 2007), Dompu (Sumbawa, November 25, 2007), Simeulue (February 20, 2008), West Java (September 2, 2009), West Sumatra (September 30, 2009), and Aceh (July 2, 2013), it is high time to do some introspection with regard to “what is the Indonesian earthquake problem”.

It must be admitted that since one of the largest tsunami in modern history December 26, 2004 in Aceh and the repeated earthquakes in the last two years, there is no drastic change in earthquake related matters, such as the enforcement of seismic resistant buildings all over Indonesia. Regulations related to earthquakes resistant buildings was only revised in 2012 after 10 years. After the Aceh (2004), Yogya (2006) and West Sumatra (2009) earthquakes, most of the buildings are still being constructed following the old practice, prevailing prior to the occurrence of the damaging earthquakes.

Normally, after those damaging earthquakes, the government should have a comprehensive plan related to earthquake resistant development. Also, until today no requirements have been issued related to retrofitting of buildings. As an irony, Indonesia enforced a compulsory primary education, on the contrary many school buildings collapsed in past earthquakes.

The damages that occurred in Yogyakarta, West Sumatra, Bengkulu, Dompu, Simeulue, West Java, West Sumatra, and Aceh showed that simple houses collapsed claiming human lifes. In Yogya, after the 2006 earthquake, JICA introduced a Building Permit System (BPS) for simple houses in villages; however, so far the code for non-engineered constructions is not yet developed. A BPS is part of an enforcement to build earthquake resistant houses.

It is very common that every stakeholder in earthquake matters tends to think that its role is the most crucial in addressing an issue. Therefore, there are always differences of opinion between scientists, geologists, engineers, administrators, social scientists and NGOs on how to solve the problem. Some stakeholders said that mass awareness campaigns are needed to create a demand for safe buildings; others said more seismic instruments are critical; or recommended tsunami early warning system and drill exercise; or suggested to make seismic micro-zonation. Administrators explain that everything is taken care off and many other statements, opinions. All those different opinions are important, however, no one can argue that the main problem is that there are millions of non-engineered houses that are not earthquake resistant (Boen, 2008).

It is very clear, unsafe building stock, particularly non-engineered constructions are the problem and the solution is to:

- ensure that all new non-engineered constructions are earthquake-resistant, and
- all existing non-engineered constructions are made earthquake resistant over a period of time through well thought simple retrofitting methods, affordable, replicable, suiting the local culture

Below are some important issues for Indonesia to ensure that all non-engineered constructions are earthquake resistant.

2.1.1. Public Awareness

Large number of losses during earthquakes in Indonesia is mainly caused by lack of public awareness and knowledge on earthquake and safer housing construction (Center for Disaster Mitigation Institute Technology Bandung, 2011). In most surveyed areas the level of earthquake awareness among local residents was very low (Building Research Institute, 2006; JICA Manado Survey Team, 2009; JICA - Aneka Asia Buana, PT, 2012). This presents an obstacle to vulnerability reduction efforts by outside agencies because the importance of the issue is not perceived by home owners. In essence it is not a priority. There are many more pressing issues in the daily lives of rural families as well as urban shanty towns dwellers and, unless the area has been struck recently by an earthquake causing substantial damage, people may not feel the need to make safety related improvements. It is essential that public awareness programs be conducted in a way that will reach rural and urban shanty town populations are effective in communicating the issues (Boen, 2005).

Information concerning the need to build earthquake resistant buildings, to use good quality materials and to adopt earthquake resistant features must be continuously and consistently disseminated to the community. Local craftsmen are perhaps the most important target group for public awareness efforts. Since they are responsible for the construction techniques, it is essential that appropriate information, as well as intensive training programs reach this segment of the rural as well as urban informal settlement's population (Boen, 2005). This shall be made as the main target of the government, to create awareness for the need to build earthquake resistant non-engineered constructions (Boen, 2008). Awareness of the need for house owners to incorporate earthquake resistant features in buildings as they are constructed. This activity will require a variety of commitments and adjustments on the part of the government. There must be a political will from the government side to make it happen.

One of the important part for a successful implementation of disaster risk reduction is that the government must have a clear understanding about perception of risk. Meaning to understand the earthquake risk and how it relates to other daily life risks. With such change in understanding, supporting activities to improve this type of housing and developing a public awareness program concerning earthquake resistant construction practice can be sustainable. (See Section 2.2.6 for more detail)

2.1.2. Technical Competence

Fifty years ago, after returning from study at IISEE, Tokyo; the author introduced the subject earthquake engineering into the curriculum of the Civil Engineering Department of one University in Jakarta and currently almost all civil engineering departments in Indonesia teach the subject. The subject is only taught at the undergraduate level and is focused on engineered buildings. Non-engineered constructions are not being taught and the subject building construction is lately not taught intensively. In the last twenty-five years, many engineers, architects, government administrators, contractors got their post graduates in earthquake engineering at various reputable universities abroad. However, to date, similar damages to non-engineered constructions are still occurring after every strong earthquake, implying that some rethinking must be introduced with regard to earthquake engineering in Indonesia.

Besides the non-performance of engineers in this regard, there has also been a considerable decay in the capabilities of artisans and technicians associated with building trade. A mason today has far lower competence than two decades ago (Boen, 2008).

Until now there are very few architects or engineers who pursue and commit to learn the non-engineered constructions because most of them will not receive adequate material rewards and must even make sacrifices. It can be seen from the amount of literatures that discuss the earthquake engineering problems for non-engineered constructions are only less than 5%, whereas the victims caused by the collapse of buildings will be more than 60% (Boen, 2000).

Not many engineers have the capability to do structural analysis for non-engineered people's houses, and even already forgot about the correct way of laying bricks, mixing concrete, preparing correct reinforcing detailing for seismic resistance. This unfortunately resulted in the poor quality of the houses built so far in Aceh and Nias that were supervised by engineers and architects who were supposed to have the very basic skills needed for earthquake resistance. Equally true is the fact that currently it is very difficult to find artisans who still have the qualifications to appropriately construct a simple house.

No doubt non-engineered constructions must be introduced in the earthquake engineering syllabus and the subject on building construction must be refreshed. However, a lot more remains to be done to raise the competence of engineers and architects regarding non-engineered constructions. It is essential for the successful implementation of improved construction practices for earthquake resistance that engineers and architects be familiar with these requirements. Therefore, the competence of engineers and architects entrusted with design and supervision of non-engineered constructions should be upgraded. Equally important is to introduce non-engineered construction in the curriculum of technical high schools and also to raise the competence of construction workers.

It is also suggested that professional societies should work closely with universities to develop courses of study for engineering and architectural students related to effects of earthquakes on non-engineered constructions. The main cause for poor quality control is that there is a gap between knowledge and application and that despite of our experience from numerous earthquakes, and the growth of our knowledge of earthquake resistant design, the principles are not being communicated to the humble local builders and craftsmen (Boen, 2001).

Many research results are not translated into action. There is a problem in information dissemination or maybe the information available is not practical, not feasible to be applied. It is apparent also that the common mode of dissemination for findings, such as journals, technical reports, conference presentations, face to face discussions etc., is not producing enough initiative of rethinking or at least being filed for future consideration (Boen, 2002).

Almost all disciplines within the physical sciences, social sciences, engineering and some biological sciences are needed. The professionals, such as seismologists, and city planners, tend to associate with each other and read almost exclusively from publications within their own discipline or trade.

It is essential that information on improving building designs to better resist earthquakes be made available to engineering and architectural students. This can be done by holding seminars, workshops, special lectures and formal courses (Boen, 2008) to exchange detailed information and to argue the merits if various perspectives, but they need to be attended by professionals from the full range of disciplines and by policy makers from various levels of government (Boen, 2002).

There are a variety of ways to encourage a community to improve its construction techniques: by giving an incentive (limited funding support) to those who build their houses according to earthquake-resistant standard design; by giving training under skilled supervision, learning-through-doing in community projects; by providing technical assistance and advice at the point it is most needed and effective on-site; by training builders at a local training center to improve their earthquake-resistant construction skills, engaging the builders in practical exercises and building sample buildings under skilled supervision.

The main parties involved in disaster risk reduction are people on one hand and the government on the other hand. People are organized at various levels in the community, in non-governmental, formal and non-formal organizations. In Indonesia to-date, almost all trainings with regards to disasters were conducted by Universities, NGOs, local as well as foreign, International agencies and without actual participation of the government.

The main objectives of risk reduction training programs are raising awareness, skill development, process training, and self-realization. Raising awareness is the most obvious goal. Disaster risk reduction must also be seen as a sequential process of development, understood by all parties in the disaster process. Disaster training must also aim at the development of specific abilities necessary to execute disaster risk reduction projects. Training must also aim at self-realization and capacity building in management on the local level (Nimpuno, 1992).

2.1.3. Guidelines for Non-engineered Construction

It must be admitted that since one of the devastating tsunami in modern history December 26, 2004 in Aceh and the repeated earthquakes in the past years, there is no drastic change in earthquake related matters, such as the enforcement of seismic resistant buildings all over Indonesia (Boen, 2008). Regulations related to earthquakes resistant buildings was changed in 2012 only, after 10 years. In Aceh, Yogya, and West Sumatra after the 2004, 2006 and 2009 earthquakes respectively, most of the buildings are still being constructed following the old practice, prevailing prior to the occurrence of the damaging earthquakes.

Similar to most of the countries affected by earthquakes, Indonesia does not have building code provisions for earthquake resistant design or non-engineered construction, even though in Indonesia, guidelines on “how to do it” for earthquake resistant as well as for reconstruction and repair of non-engineered constructions are available since almost 35 years ago (Boen, 1978). In countries in which building codes exist, the code provisions are in many cases inadequate. The promulgation of codes applicable to specific types of non-engineered structures would significantly contribute to achieving better resistant structures built in accordance with local needs and capabilities. The code should be a performance code rather than a specification code. The code should be more an expression of desired results than a set of instructions on how to attain that. It should be a “how to do it” manuals and guidebooks types on how to build an earthquake safe house and retrofitting.

The code also should be life safety rather than property safety oriented. Earthquake resistant design codes should not be intended to ensure against damage to structures. It should be assumed that a large earthquake will cause heavy damage, but it should be our intention that it will not cause building collapse with consequential loss of life and injury.

Minimum building standards based on building performance and emphasizing the safety of the occupants should be developed by the government for non-engineered constructions. For the purpose, all existing/available materials should be adopted and not try to re-invent the wheel. Learning from the reconstruction of houses in Aceh post-earthquake and tsunami December 26, 2004, relevant guidance for good practice of non-engineered constructions for new construction exists or has been developed for Indonesia, long before the reconstruction in Aceh and Nias (Boen & Priyono, 2011). However, lack of coordination and leadership within the shelter sector resulted in that these good references were not widely distributed and were frequently either not known about. All those relevant materials available during the reconstruction of Aceh were ignored and instead, many foreign consultants made their own layout and adopted the confined masonry construction method, but leaving out the detailing for seismic resilience. (Ove Arup & Partners Ltd., 2006; Ove Arup & Partners Ltd., 2007). Below are existing non-engineered guidelines in Indonesia to 2013 (Figure 9):

(a) 1978, Manual Bangunan Tahan Gempa, (Detailer’s Manual for Small Buildings in Seismic Areas), Boen.
(b) 1980, IAEE Monograph Non-Engineered – Guidelines for Earthquake Resistant Non-Engineered Construction, Arya, Boen, Grandori, Bonedetti (alternate), Grasex, Poliakov, Moinfar, Umemura, Ohsaki (alternate). Latest version in 2012 by UNESCO & IISEE.
(c) 2005, Poster Minimum Requirement for Earthquake Resistant Masonry Building, Boen.
(d) 2005, Constructing Seismic Resistant Masonry Houses in Indonesia, Boen.
(e) 2006, Manual Bangunan Rumah Rakyat Tahan Gempa (Barrataga), UII.
(f) 2011, Seismic Design Guide for Low-Rise Confined Masonry Buildings, Meli, Brzev, Astroza, Boen, Crisafulli, Dai, Farsi, Hart, Mebarki, Moghadam, Quiun, Tomazevic, Yamin.
(g) 2012, Persyaratan Pokok Rumah yang Lebih Aman, Boen, Suprobo, Sarwidi, Pribadi, Irmawan, Satyarno, Saputra (published by JICA).
(h) 2012, Key Requirement for Safer House, Boen, Suprobo, Sarwidi, Pribadi, Irmawan, Satyarno, Saputra (published by JICA).

Figure 9 – Existing Non-Engineered Guidelines

If codes for non-engineered construction already exist, the code must have clout, because codes are of little use unless it is backed by a powerful enforcement agency and a comprehensive inspection service (NBS Building Science Series 106, 1976). Effective communication of correct technics for earthquake resistant houses is essential. Although the technology may be known by engineers/architects and those involved in housing development, simple materials, easily understandable to the villagers must be developed and disseminated (Boen, 2008; NBS Building Science Series 106, 1976). Simple, "how to do it" guidelines for construction of new buildings as well as the repair and strengthening of existing buildings must be increased and disseminated.

Code enforcement for non-engineered as well as engineered buildings is in many cases unsatisfactory, mostly because qualified personnel are not available. It is suggested that local credit-granting institutions be responsible for inspecting the building construction they finance and make loans conditional upon compliance with building codes. The enforcement of provisions applicable to local non-engineered construction could be entrusted to properly trained inspectors with limited formal education. It is stated that building codes should not obstruct technological innovation. As an example, in Aceh and Yogya, one of the major constraints found was the lack of understanding to adopt construction methods for earthquake resistance. Among the contractors, engineers, architects as well as construction workers, little or no awareness of earthquake risk exists; therefore, attempts to introduce new practices did encounter difficulties (Boen & Priyono, 2011).

Due to the long interval between events, even those who experienced earthquakes often felt the threat was too remote to warrant change. House owners were aware of seismic activity, however, tremors of recent memory failed to drastically affect their houses, they felt that their houses were strong enough to withstand earthquake shakings. Construction habits will be dictated by tradition, popular trends, availability and cost of labor and materials. Concurrently, efforts to enact legislation against erratic building habits in villages should also be encouraged (Boen, 2008; JICA - Aneka Asia Buana, PT, 2012; Center for Disaster Mitigation Institute Technology Bandung, 2011).

Efforts must be emphasized on how to make such masonry houses earthquake resistant and information dissemination on how to appropriately build masonry houses, meaning the enhancement of the current practice to produce good quality buildings as a culture (Boen, 2005).

Judging from the huge number of houses all over Indonesia that must be retrofitted, it is very urgent to adopt documents on seismic assessment of existing non-engineered constructions, and to introduce simple, replicable, affordable method for seismic retrofitting.

2.1.4. Non-engineered Construction considered as Informal Sector

In Indonesia, most of the common people's housing construction occurs in the informal sector. The informal sector is that which is outside the legal and banking systems and is therefore unregulated and uncontrollable. It is beyond the applicable rules and regulations and the government agencies do not care. Those houses are registered for land tax purposes only. Therefore, most of the informal sector do not have the necessary permit, are allowed to grow continuously until it cannot be controlled. This is one of the reasons why many houses are damaged or collapsed during earthquakes and caused many casualties.

This problem cannot be neglected and it is time for the authorities to give a simple and practical knowledge how to build an earthquake resistant "house" to the communities in areas prone to earthquakes. The communities generally tend to be more concerned with how to survive day by day rather than to build earthquake resistant houses. Usually the authorities only allocate the funds to develop the formal sector and neglect the informal sector. It is a mistake if people's housing construction is isolated from the comprehensive strategy to increase the earthquake resistance of buildings.

In Indonesia, informal housing sector also exist in urban areas, namely the squatter settlements or shanty towns and that represent some of the highest risks of life-loss, injury, homelessness and emergency needs in the event of an earthquake. Earthquake protection for these areas has to be part of a general retrofitting strategy.

2.1.5. Research and Development

In Indonesia, with regard to non-engineered construction, there is a clear need to focus research on "engineering" of earthquakes as against the focus on "science" of earthquakes that many researchers have been doing. It is important to put in perspective that earthquake safety is a rather challenging engineering problem requiring decades of focused work, and that even though science is important, engineering aspects shall not be ignored. The contributions science can make to reducing earthquake disasters are necessary and needless to say that if possible, the best approach to earthquake problems is to work on all the fronts simultaneously: engineering, science and instrumentation, public awareness, public policy, etc. (Jain, 2005; Boen, 2008).

2.1.6. Local Social, Economic, and Cultural Problems

Social and economic factors affecting housing construction in developing countries include: shortage of funds, heavy migration of rural population to urban centers, population growth, and markets of insufficient size to insure economies of scale adopt a more organized construction methodology, insufficiently developed transportation and distribution systems, shortage of skilled labor, generally low standards of workmanship. An additional negative factor is users' resistance to certain construction materials and systems. Such resistance can in many cases be overcome.

One of the main challenges is reinforcing cultural continuity through development opportunities that are afforded through post disaster rehabilitation, so that one does not end up with cultural incompatible solutions, which prove unsustainable in the long run. Moreover cultural continuity and compatibility needs to be considered in the vital aspects of 'earthquake safe' technology transfer during post-earthquake reconstruction. There exists critical relationship between technological knowledge, and the qualitative aspects related to community relevance, social acceptance etc. besides economic viability and long term sustainability.

Programs must be in accordance with the social, economic and cultural reach of the community/population it is to benefit. This is important to assure that the program penetrate the culture and therefore will be adopted and acquired and learned by the people. (Boen & Jigyasu, 2006).

2.2. Obstacles in Disaster Risk Reduction

In Indonesia, it seems that the term disaster is generally interpreted as a misfortune or calamity and anything of a distressing nature. Therefore, different users interpret disaster in different conceptions. An earthquake engineer defines disasters as plate movement and damage of buildings. The politician looks at the political consequences and the NGOs in relation to relief needs etc. Such interpretations were based on the characteristics of the physical forces resulting in damages. A clear example is the preparedness efforts for tsunami were mainly concerned with installing early warning equipment and scientific study of the physical phenomena (simulation of tsunamis). It is time to pay attention to the inherent social and human dimension in each disaster.

Indonesia as an economically developing country with many competing demands on available resources, the government is experiencing difficulties to justify the time and resources needed for earthquake risk reduction. One of the causes is that there is wide spread perception among the government that the immediate cost for mitigating earthquake risk is big. Such perception has been spread by technical community who knows only half half about earthquake resistant design. They play safe and exaggerate the design. In fact, the additional cost to do it right the first time for non-engineered construction is insignificant, it is a matter of quality materials and workmanship. The quantities of materials used are the same.

2.2.1. Current Disaster Risk Reduction Policy in Indonesia

Indonesia has accumulated a large pool of earthquake engineers in academic as well as governmental organizations who are supposed to contribute to the development of science and engineering of earthquakes. However, until today, every time there is an earthquake, people's houses and school buildings are damaged or even collapsed and caused casualties, meaning the earthquake risk in Indonesia is not much improved than they were 40 years ago (Figure 6 and Figure 7). Why could this happen? With considerable research and guidelines/manuals regarding non-engineered construction available in Indonesia, why then do we continuously experience damages of non-engineered construction? Will the last earthquake experience be repeated in the next earthquake?

As mentioned in Chapter 2.1.6, earthquake risk reduction is related to social, cultural, political issues and this resulted in lack of the necessary organizational infrastructure. Experts in Indonesia are mostly interested in engineered construction and do not have a clear understanding that for non-engineered construction, one must adapt foreign technologies and must develop appropriate technologies workable and replicable for the common people. Most experts are applying "One Size Fits All" strategies for risk management in various countries; they are copying "modern" technologies and risk reduction options that are not in accord with the social, cultural and economic structure of the common people in each respective areas. This always ends up in a "One Size Fits None Very Well".

After every earthquake, "experts", government officers, universities are very good at arriving at the causes of disaster, injured, death, economic losses etc., and the community is lead to believe that their earthquake safety is taken care of, until the next earthquake shows that not much has been done since the last damaging earthquake. To be able to find the answer to the above questions, one must review the disaster management in Indonesia.

2.2.1.1. Disaster Risk Reduction Policy before the Aceh, December 26, 2004 Earthquake and Tsunami

Before the catastrophic earthquake and tsunami of Aceh, December 26, 2004 which caused thousands of lifes and caused the devastating damages and material loss, disaster management in Indonesia was emphasized in emergency response only with only little or no emphasis on disaster risk reduction. Standards meant to ensure public safety, such as regulations for building permits, utilization of surrounding land, spatial management, and so on, did not contribute substantially to reducing disaster risks. There is no policy on disaster risk reduction and all activities are isolated in a vacuum of statutes with no specific guidelines for activities related to disaster management (IRG-Tetra Tech Joint Venture for reviewed by the USAid, 2007).

By Presidential Decree no 43 1990, BAKORNAS PB – Badan Koordinasi Nasional Penanggulangan Bencana (National Coordination Board for Disaster Management) was established.

BAKORNAS PB, as mentioned earlier, is not a typical government agency but a coordination body comprising of a council of ministers, headed by the Vice President, and assisted by a secretariat. The BAKORNAS PB was seen more as performing secretarial tasks, and not as much a coordination body.

The regional governments have similar structures for coordination called SATKORLAK PB (Coordinating and Implementation Unit for Disaster Management) at the provincial levels and SATLAK PB (Implementation Unit for Disaster Management) at the district or municipal levels. SATKORLAK PB and SATLAK PB activities are to be financed by provincial and district / municipal budgets. The institutional structures in Indonesia are designed for emergency response, with very little or no emphasis on disaster risk reduction.

Due to these facts, it is obvious that during the BAKORNAS era, disaster risk reduction was not known and this is the cause that materials for disaster risk reduction of non-engineered constructions were NOT promoted / disseminated.

2.2.1.2. National Disaster Risk Reduction Policy since 2007

After the December 26, 2004 earthquake and tsunami in Aceh, the Government of Indonesia recognized that its national disaster plan was grossly inadequate to effectively respond to a major catastrophe and should be reformed. Disaster management cycle must be regulated and managed through an integrated, coordinated and comprehensive disaster management plan system.

Therefore, the Disaster Management Law 24/2007 was established on April 26, 2007 with the aim to reduce disaster risk and incorporates disaster risk reduction in its development plan. The new law clearly recognizes the shift in paradigm from a focus on Disaster Response (DR) to enhancing Disaster Risk Reduction (DRR) while clearly identifying a systematic approach to disaster management across the three phases of the disaster management cycle, pre disaster, during a disaster and post disaster (Center for Excellence in Disaster Management & Humanitarian Assistance, 2011; Safe Communities through Disaster Risk Reduction in Development Programme (SC-DRR), 2011).

As mandated in Disaster Management Law 24/2007, the National Disaster Management Agency (Badan Nasional Penanggulangan Bencana – BNPB) was established by Presidential Decree No. 8, January 26, 2008, as a non-departmental Government Institution on a level equal to a ministry and reporting directly to the President. Note: as a matter of fact, the translation of “Badan Nasional Penanggulangan Bencana” into “National Disaster Management Agency” is wrong. The word “penanggulangan” is not “management” but more appropriate “prevention”. Therefore, all phrases “disaster management” are a misnomer.

Subsequent to which the Minister of Home Affairs issued Decree No 46/2008 requiring the establishment of Local “Disaster Management” Agencies (BPBDs) in all provinces by the end of 2009. The national government made it mandatory for BPBDs to be established in every province and hence the provincial governments have a budget line for “disaster management”. However, until December 2013, there are some provinces where BPBD are not yet established.

Conceptually the content of the law is good enough to make disaster risk reduction as a culture in Indonesia; HOWEVER, the understanding and realization of disaster risk reduction in almost all ministries are limited if not none.

From observations in various provinces and districts, the author is of the impression that so far, BPBDs do not realize that they are not SUBORDINATES of BNPB. Law 24, 2007 stipulates that BPBDs are sub-COORDINATEs of BNPB. Therefore, BPBDs are responsible in developing policies and implementation of disaster risk reduction in their respective areas.

Indonesia is highly populated and many places are difficult to reach due to geographic condition. Therefore, many people will suffer more during disasters. Apart from that, the National “Disaster Management” Agency (Badan Nasional Penanggulangan Bencana – BNPB) as well as Local “Disaster Management” Agencies (Badan Penanggulangan Bencana Daerah – BPBDs) are not fully functioning and many are still deficient. In other words, there is clear indication of “failure in disaster management”.

Some problems of “disaster management” in Indonesia are as follows:

2.2.2. Un-optimal Performance of Disaster Risk Reduction

The performance of “disaster management” in Indonesia is still unoptimal. The government, the community and all relevant “disaster management” stakeholders in Indonesia have not been prepared to deal with disasters so that the number of disaster victims every year is still high with huge material losses caused by disaster. The coordination and cooperation in emergency response and post-disaster recovery are still not optimal (National Agency for Disaster Management, 2010).

Data concerning the number of dead and injured were different from time to time and made it difficult to allocate medical personnel and equipment, including medicine needed to treat disaster survivors. Likewise, data about houses totally destroyed, heavily damaged and lightly damaged, public facilities and infrastructures are seldom consistent and sometimes there are several versions of data that conflicts with each other. This will further slow-down the overall recovery of the disaster affected communities.

The institutional orientation of “disaster management” in Indonesia still tends to emphasize more on emergency response rather than disaster prevention and risk reduction. It seems that the understanding and realization that disaster risks may be reduced through development interventions are still very limited. Disaster Management Law 24/2007 has shifted “disaster management” paradigm from a responsive orientation (focused on emergency response and recovery) to a preventive one (risk reduction and preparedness), however in implementation there are still few disaster risk reduction programs that are planned and programmed. It seems that BNPB lacks professionals in the various disaster related fields who clearly understands what must be done and how to implement the disaster risk reduction programs.

Mitigation and prevention are the mechanism to break the cycle of repetitive damage and redevelopment. Therefore, the most effective approach to reducing the long term impact of disasters is to incorporate mitigation activities into the process of development planning and investment project formulation. What is often called reconstruction/recovery process after a disaster is development in and of itself. Unfortunately this is not happening in all recovery programs so far!

In most developing countries, the reason why mitigation is not receiving attention is the fact that the country is immersed in an economic crisis of such proportions that the institutional sector is primarily concerned with its own survival with little or no time or resources for disaster prevention and mitigation.

As is widely recognized, earthquakes are not, in and of themselves, disasters, but are agents that transform a vulnerable condition into a disaster. In developing countries the condition of vulnerability is a result of its state of poverty, caused to a large degree by the following factors: debt crisis, population growth, mass urbanization, and political instability.

In settlements areas exposed to hazards, risk must be divided into acceptable risk and unacceptable risk. Acceptable and unacceptable damage should be defined. Collapse, extensive structural damage is unacceptable. Slight structural damage is acceptable since there is no “earthquake proof” building. What is acceptable is to make the probability of failure smaller. Decision makers must convince the public concerning what is acceptable risk and what unacceptable risk is and subsequently decide the coping methods for those categories of risks.

Risk reduction measures need to be absorbed in the development programs of Indonesia. All measures should be absorbed into building practice and public awareness, causing a major reduction in vulnerability. Measures adopted should be linked as closely as possible to the risk identified of the vulnerability of population affected.

Below are several cases that clearly indicate the un-optimal performance of BNPB as coordinator of “disaster management” in Indonesia:

2.2.2.1. <u>Post Disaster Risk Reduction Opportunities in Aceh</u>

First of all, it is a fact that BNPB was only established January 26, 2008, therefore, most of the un-optimal results of the rehabilitation and reconstruction of Aceh was during the Bakornas and Aceh Rehabilitation and Reconstruction Agency era. However, BNPB could have introduced disaster risk reduction measures in the last year of the reconstruction of Aceh which terms of assignment ended April 2009.

In Aceh, with the unprecedented generosity in solidarity of the world - governments and people with approximately US$7 billion fund for rehabilitation and reconstruction, there is no reason not to implement measures which can eliminate or at least reduce vulnerability to earthquake effects. As a matter of fact, in the Aceh case, with the available fund, the following three opportunities may arise to intervene in the course of earthquake mitigation, namely, the first is to take necessary actions to prevent before the disaster occur, all new structures must be made earthquake resistant. Human memory is short and most of the people in Aceh already forgot or do not realize that two segments of the Sumatra fault run under their city Banda Aceh and fails to take action to prevent it.

The second is mitigation before the effects of an earthquake become significant, before the fault breaks, all buildings built so far could be at risk and must be assessed and retrofitted if needed. Most of those buildings did not follow the seismic code. This type of action could be subject to rejection by building owners since people's perception of the danger probability is low and the cost for retrofitting could is high. Retrofitting or replacing buildings, being a corrective measure, could be costly but that is the appropriate approach to save lives.

The third is mitigating earthquake hazards after serious effects of the earthquake have been experienced. The third opportunity is relatively easier to implement. The reasons are that people have seen the consequences of the earthquake and the decision makers as well as public perceptions of vulnerability and risk are very high. Therefore, the occurrence of an incident creates substantial political support on top of demand for public action to remedy hazardous conditions. Such a cause and effect relationship has spawned most hazard reduction legislation. Such opportunity shall be fully utilized during the reconstruction. With regard to the reconstruction of houses in Aceh, it is apparent that none of the above mitigation opportunities were implemented. Therefore, the reconstruction of houses in Aceh missed the golden opportunity to introduce long term mitigation measures, retrofitting of existing undamaged buildings and introducing correct seismic safe construction requirements for new buildings.

Figure 10 – Houses under Construction in Aceh, Poor Quality Materials and Poor Workmanship

Most of the houses built are not earthquake resistant, while earthquake resistant construction of buildings are examples of measures that can increase the capacity of facilities to withstand the impact of earthquake hazards. Measures such as zoning ordinances, insurance and tax incentives, which direct uses away from hazard prone areas, lead to impact avoidance. For Banda Aceh, due to the fact that two segments of the Sumatra fault are located under the city, the most viable choice is that all buildings can withstand the impact of earthquakes, meaning all buildings must be made earthquake resistant.

World organizations and the Aceh Reconstruction & Rehabilitation agency produced excellent and comprehensive reports which can be accessed from their websites, however, only very little if none is disclosed with regard to the quality of the approximately 127,400 houses built until December 2008 (Boen, 2009). As written in some (very few) reports and papers, most of the houses built are not earthquake resistant and the February 20, 2008 Simeulue earthquake confirmed the above statement. Poor quality of materials and workmanship, resulted in very poor quality houses is also acknowledged briefly in some reports. In that report there is some mention about the slow pace of retrofitting of newly built houses by introducing structural improvements, which is a very clear indication that many newly built houses are not seismic resistant. It would be appropriate if all parties involved in the reconstruction of houses publish similar reports and explain in greater details the quality of the built houses. If retrofitting was introduced, it should be mentioned what methodology was applied in assessing poorly built houses and the subsequent introduced structural improvements. This is important since in Indonesia, the sophistication required for undertaking retrofitting has not been adequately articulated, particularly for Indonesian non-engineered construction (Boen, 2008). In short poor quality houses are a sign that mitigation options are not adopted.

The main reasoning why mitigation opportunities were ignored could be that the reconstruction of the houses in Aceh is heavily dependent on external resources, financial as well as personnel and such heavy dependency caused a loss of local control. Loss of control can be seen from the fact that all NGOs and other donors develop their own guidelines due to non-availability of mandatory procedures to regulate the design and quality of housing that were built, all organizations involved in the reconstruction of houses in Aceh and Nias developed their own guidance (Boen, 2009). It is apparent that the authorities did not organize themselves appropriately in order to use wisely resources and skills as they are offered and at the same time to resist unneeded or unwanted supplies, personnel, experts, and advice.

2.2.2.2. *Early Warning System of Mentawai Tsunami, October 25, 2010*

Several minutes after the Mentawai October 25, 2010 earthquake, Badan Meteorologi, Klimatologi, dan Geofisika (BMKG) issued the tsunami early warning. However, the tsunami early warning was cancelled since BMKG perceived that the possibility of tsunami has passed. Several minutes after the cancellation, tsunami swept 77 villages in Mentawai islands and caused 400-500 victims (Science Daily - Science News, 2010).

Furthermore, BNPB has confirmed that there were no sirens installed on the Mentawai islands (Speiden, 2010). A spokesperson for the BNPB, said the organization was aware of the lack of sirens on the Mentawai islands, while the head of Indonesia’s Tsunami and Earthquake Centre said that the system worked according to plan.

After the Mentawai tsunami, the TEWS team issued a statement that “The Mentawai quake also showed the limits of any tsunami warning” (Science Daily - Science News, 2010). It seemed that the TEWS was passed off as a learning experience at the expense of hundreds of lifes.

Poor emergency preparation increases the number of victims than would otherwise have suffered in the presence of better prevention (Kuntjoro & Jamil, 2010). Therefore, BNPB needs to work a lot harder in its efforts to work with local governments and citizens to get a better system in place on the ground.

2.2.2.3. <u>Reconstruction of Houses in West-Sumatra post the September 30, 2009 Earthquake</u>

In October 2009, one month after the West Sumatra earthquake, BNPB agreed to provide assistance to house owners, with a maximum of Rp 15 million for “heavily” damaged houses. The ministry of Public Works was assigned to hire facilitators to provide technical assistance on how to rehabilitate their houses and at the same time act as project manager to issue certificates of payments based on the progress of rehabilitation of the houses. The department of Public Works started hiring engineers and was given short trainings on how to assist house owners in rehabilitating their houses apart from training in certification.

Figure 11 – Abandoned Damaged Houses – Owners did not Retrofit Their Houses even though They Have Received the Funds – Picture taken February 2013

Apparently until 2011, the fund was not yet disbursed. The hundreds of facilitators were paid but could not work and after approximately 5 months were terminated. This was clearly a waste of time as well as fund and bad coordination. Only in 2012, the fund was available but it was decided that it will be paid in 3 installments, 5 million each term. The disbursement was not all at once but was gradually districts by districts. The last fund was disbursed in February 2013 and many complain that the payments were not the full amount and parts were "taxed" by facilitators as well as village chiefs. This was reported in the media. A clear indication of non-performance can still be seen today among others in Pariaman area (see Figure 11 taken in February 2013) where many of the house owners already received the full amount from the government BUT used for other purposes and their houses are still untouched and same damage condition as in September 2009. How can the owner receive payment while the installments can only be paid based on the progress of work?

There was also a clear indication that the damage assessment was not done properly. Many of the damaged houses were demolished and the owners build new houses far bigger than before the earthquake (Figure 12); meaning that the owner has enough funds and does not need to be supported.

Figure 12 – Lack of Damage Assessment: New Houses Built Bigger than the Damaged Houses due to Earthquake – Picture taken January 2013

2.2.3. Lack of Public Awareness

Many useful programs in various parts of Indonesia related to disaster risk reduction measures are not continued after the projects ends, or in other words, they only last as long as the project last. A very valid example is the reconstruction of houses in Aceh after the December 26, 2004 tsunami. After the tsunami disaster, both government and private individuals share a common interest in reducing future loss of life, injury, and property damage. Where structures have been totally destroyed, there is opportunity to start over and "do it right" to accomplish not only hazard reduction goals but broader land planning and economic development objectives (Boen, 2009).

However, despite of the severe impact of this disaster on the economy and well-being of the Aceh province, it seemed that relatively little is being done to prepare for, prevent, or mitigate the effects of future earthquake events. Most probably this is due to misinterpretations and misunderstanding with regard to that the interval between events is long and therefore should receive little attention. Probably also, all attentions are focused on the tsunami-centric planning namely to structure town and village spatial plans for the eventuality of another tsunami. Earthquake disaster reducing option that is considered as top priority by the authorities in Indonesia is installing tsunami early warning system. Tsunami early warning is about imminent dangers that warrant emergency measures such as alertness and evacuation. On the contrary, the segment of the Sumatra fault that runs from Lampung to below Banda Aceh is a warning about potential danger that warrant preventive measures, namely mobilizing disaster reduction actions.

It seemed that there is a lack of understanding that disasters (in contrast to hazard) can be prevented, the impacts of earthquakes can be mitigated, and that mitigation measures can be incorporated into the reconstruction planning.

Judging from the fund available for the reconstruction of houses and the mitigation opportunities that were ignored in the Aceh reconstruction case, it is reasonable to assume that there is a lack of proper knowledge and understanding concerning disaster risk reduction, lack of proper knowledge that mitigation is the mechanism for breaking the cycle of repetitive damage and redevelopment, and for preventing unwise development. This is because a building that withstands earthquakes means that it represents a safe refuge for its occupants; eliminates the tremendous discontinuity and economic burden; saving of building materials; saving of financial resources and for the government; it represents reduction of further strains on a reconstruction economy, thereby enabling the policy makers to spread financial resources.

For example, the seismic code for non-engineered construction which was drafted for the reconstruction of the houses in Aceh was not managed properly resulting in poorly built houses. This can also be witnessed from the extensions to the original houses constructed by many recipients. It is doubtful whether the newly extended houses were re-analyzed because the new extended house will behave differently than the original house (Boen, 2008). Another fact indicating the building code is not imposed is that all new construction follows the old habits of the pre December 26, 2004 earthquake without taking into account seismic detailing. Instead until today the understanding among stakeholders of the reconstruction is that the cost is high for achieving seismic resistance for new or existing buildings.

A record number of workshops, trainings related to non-engineered construction were conducted and yet, as is well known, almost all houses built within the framework of reconstruction are not earthquake resistant. Eight years after and even during the reconstruction period, new houses and shop houses were built based on the old habits used before the 2004 earthquake and tsunami. Meaning, good practice was not absorbed and life goes on as usual. The Simeulue earthquake of February 20, 2008 and the Aceh Tengah / Bener Meriah earthquake of July 2013 were evidences where newly built houses were severely damaged or collapsed.

2.2.4. Lack of Professional Accountability

One other important aspect which contributes to the lack of progress in earthquake risk reduction for non-engineered construction in particular is the lack of professional accountability for poor performance of non-engineered construction and poor practice of officials in charge of the problem. The main reason is very poor "law enforcement". Laws are restrictions for the government as well as the communities, however, law is important in setting safety standards and law is a vital element in public education. As an example, most of the building materials sold in Indonesian markets are not in accordance with the Indonesian standards for each respective material. The worse, the courts are not transparent. Structural failures were not widely published, particularly when government officials and giant contractors are involved.

To enforce the law for countries with deteriorated conditions will take a very long long time. In the meantime, life must go on and many more houses are being built as "usual".

In Chapter 2.1.2 it is explained that there is lack of communication among researchers, academics and the people of the country. Those experts were not able to make the people, the engineering community, government organizations and the regulations / law makers aware about the type and level of risk and what best measures should be taken that cost minimum but get the maximum results. Until today, many researchers, academics, and the few well-known professionals are exchanging views in conferences, seminars, workshops among themselves only, and unfortunately many of their works have no applicability in real practice. What is needed are topics that are foundations worth further study or useful for everyday application for the common people. From the author's observation in Indonesia, the reason that there is a lack of communication between the thinkers and the do-ers is that most researchers, academics and well known professionals are only presenting unrealistic brain teasing topics with a view setting a "clever " example or for the sake of research only.

Disasters are complex events and so are disaster prevention and mitigation. In Indonesia, among those who have responsibilities for disaster risk reduction, not everyone agrees what the subject is mainly about. The segment of disaster risk reduction has received little attention outside certain people with highly specialized scientific group, namely risk assessment and reduction through structural and non-structural measures as a function if the hazards involved. The main cause for that is many of the decision makers do not master a number of basic principles in earth sciences and engineering. To solve such problem is that knowledge must be reformatted and presented to the lay user.

Apart from the above, those who know about disaster risk reduction face many conflicting beliefs, e.g. scientists and engineers tend to assume that a scientific explanation of the underlying cause of disasters is enough to provide the basis for preventing further disasters from occurring. Besides that, many scientists and engineers assume that what is self-evident to the scientists and engineer must be self-evident to everybody else. For social workers and NGOs are of the opinion that disaster risk reduction measures is basically about a social management and policy making and pre-disaster planning is about as preparedness. Also, the subjects areas related to disaster risk reduction are too diffuse, covering broad policy issues, relating mainly to response, relief and rehabilitation.

All those assumptions are maybe not wrong, however, in disaster risk reduction measures, all must work together, since alone will not be enough. Thus, disaster risk reduction must consist of measures in the social, economic, political, scientific and technical fields.

2.2.5. Lack of Information Dissemination

As mentioned previously, this clearly indicates that many research results and training are not translated into action and that there is a gap between research, training and implementation. It might be that there is a problem in information dissemination or maybe the information available is not practical and thus not feasible to be applied??? Information available is not communicated to the community because no engineer is interested in non-engineered construction. It is clear that in non-engineered construction case, the common mode of dissemination for findings, such as journals, reports, conference presentations, face to face discussion etc. is not producing enough initiative of rethinking or at least being filed for future consideration. There might also be a communication problem among the various sciences. To deal with non-engineered construction, almost all disciplines within the physical sciences, social sciences, and engineering sciences are needed. The professionals, such as seismologists and city planners, tend to associate with each other and read almost exclusively from publications within their own discipline or trade. Workshops provide ready opportunity to exchange detailed information and to argue the merits of various perspectives, but they need to be attended by professionals from the full range of disciplines and by policy makers from various levels of government.

Almost all research works in earthquake disaster risk reduction only reach the middle class level and not the lowest level. They are in fact the ones really in need of the information and application, yet most findings are not being communicated to the users group. That it is probably not too exaggerated to say that 90% of all the earthquake hazard relevant research findings never reach the common citizen. Apart from that, many of them would have no direct application to individual citizen, but some would be useful, and there are few systematic efforts to see that the valuable information / message get to the potential users. Judging from the repeated earthquake damage, this is particularly true for Indonesia in particular and developing countries in general.

As discussed, Indonesia does not lack technical capacities for DM (IRG-Tetra Tech Joint Venture for reviewed by the USAid, 2007). For example, simple, easy to use safer building guidelines for earthquakes already existed more than three decades ago. So it is surprising that no efforts are being made to ensure that reconstruction in the wake of the three or four major disasters in the last two years incorporates an element of building back better.

2.2.6. No Actions Related to Disaster Risk Reduction

According to the Disaster Management Law 24 / 2007, the Government shall network with a number of entities such as “disaster management” agencies, research institutions, “disaster management” specialists, NGOs (AFD, USAID, JICA, UNDP, World Bank, etc.), community groups, line departments, local Government authorities and other stakeholders to augment the capabilities of all relevant entities. However, BNPB and particularly BPBD, did not take possession of what have been done by institution / NGO works. For example, JICA, one of active agency in Indonesia, has accomplished many projects from 2006 until now, such as producing and distributing for free manuals of earthquake resistant construction of non-engineered houses and retrofitting people’s houses; did surveys regarding building materials and workmanship in Sumatra, Manado; introducing Building Permit System for non-engineered houses in Yogya, Sumatra and Manado; producing tutorial videos concerning retrofitting of non-engineered houses in Sumatra. However BNPB or BPBD never tried to take possession and use all of those JICA‘s project results to enhance their disaster risk reduction programs. The community awareness of Building Permit System is very low and BPBD do not take action to disseminate the importance of Building Permit System.

Another example, since 2009 until now, the Disaster Study Center of Andalas University has accomplished many earthquake risk reduction projects by retrofitting many school buildings, people’s houses, religious buildings, and also commercial buildings in West Sumatra. However, BPBD has no interest to learn how to retrofit such buildings in order to reduce the earthquake risk, especially to disseminate those simple and affordable techniques of retrofitting buildings to resist earthquake.

In Disaster Management Law 24/2007, there is no clear definition about capacity building to cope up with any disaster, includes identification of existing resources relevant to any disaster and resources to be acquired; acquiring and creating resources, organization and training of groups in local community; and coordination of such training. BNPB is fully aware of its shortcomings, but did not do much. So far, there are almost no actions related to risk reduction, especially for non-engineered construction.

The above un-optimal performance of BNPB as well as BPBD indicates clearly that disaster risk reduction is not yet practiced and therefore, no mitigation actions are implemented with regards to non-engineered constructions (people's houses) and in spite of the fact that all the materials with regard to non-engineered already existed since 35 years ago, it can be expected that the next earthquakes might still take human life. If mitigation policy is implemented today, the impact can be seen after 20 years from now. The recent Aceh Tengah earthquake of July 2, 2013 clearly demonstrated this.

So far, prevention and mitigation mostly stressed on physical solutions, while planning is not primarily the search for the implementation of technological solutions. Therefore, it is vital to develop a societal perspective on disaster risk reduction and preparedness. Basically, earthquake disaster reduction is mainly vulnerability reduction, and that means a change in behavior with respect to earthquake hazards.

One of the important parts for a successful implementation of disaster risk reduction is to have a clear understanding about perception of risk. Meaning to understand the earthquake risk and how it relates to other daily life risks. In this respect, the experts in Indonesia have failed in raising awareness of the people about the earthquake problems and how to solve them. Most of the time, after a devastating earthquake, experts when interviewed in the media are creating "scare-ness" instead of awareness. This is because media in general are after "sensations" to attract audience/readers.

One of the issue is on how the tangible segment of disaster risk reduction measures, namely the control and reduction of the physical damage caused by sudden and violent phenomena earthquakes, flood, volcanoes, cyclones, landslides, etc. can be presented in such a way that it will be interested to disaster risk reduction managers at the technical, professional, administrative, policy making levels. For this purpose, the scientific and technical knowledge must be adapted to the needs of policy makers and administrators dealing with disaster risk reduction measures, particularly in districts and sub-districts area.

Current training programs are mainly about emergency management rather than disaster risk reduction and the information gives little attention to multidisciplinary development. Also, the current trainings are mostly top down approach. The material are sectoral, addressing specific groups such as technicians, health staff etc. and most of the materials contain about awareness raising for politicians and planners and very little about how to information.

The main cause of all what is stated above is once again related to lack of political will.

Chapter 3 Non-Engineered Constructions in Indonesia

3.1. Definition

Non-engineered constructions consist of residential buildings and commercial buildings up to two floors built by the owner, using local craftsmen, using local building materials, and without the help of architects and engineers / structural experts (IAEE, 1980; Arya, et al., 2012). The non-engineered constructions should be built with marginal cost and materials available on site.

3.2. Types of Non-Engineered Constructions

Non-engineered constructions in Indonesia can be divided into two main categories. The first category of non-engineered constructions is those built according to tradition, their types suiting the culture and materials available in that area. This is the so called "indigenous" buildings and belongs to the "fading architecture" type and is currently categorized as heritage buildings.

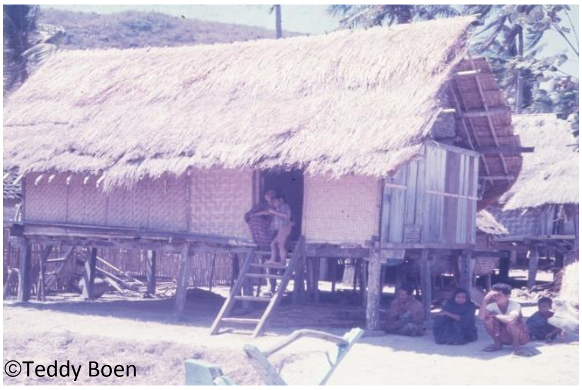
The second category of non-engineered constructions considered are single family residences and smaller commercial structures in developing countries which are built by landowners or local artisans without the benefit of engineering or architectural help.

3.2.1. Traditional / Indigenous Buildings

In the past, in Indonesia, most dwellings (non-engineered constructions) constructed in small towns and villages are built according to tradition, their types suiting the local culture and materials available in that area. This type of buildings is generally also called indigenous or vernacular buildings. Indigenous buildings are gradually fading and replaced with the second category of non-engineered constructions, namely either city type masonry construction or a combination of traditional look only but not adopting the traditional skills and crafts in detailing, material use, etc. As mentioned earlier, the rapid increase in numbers of the second category of buildings is among others due to population growth and its increasing concentration in urban areas and the prospering economic condition.

The traditional buildings generally have a good record or good performance in past earthquakes. This is due to the fact that man and nature have co-existed on the planet of earth for a long time. Since the primitive days man has tried to adjust himself to the conditions of environment and made feeble attempts to cope with the fury let loose by forces of nature. The pattern of human settlements and traditional methods and materials for traditional buildings on regional basis embody the accumulated traditional wisdom, experience, skill, and craft evolved through the ages. Some of the buildings which have existed for centuries have withstood the onslaughts of earthquakes. However, due to urbanization, the trade of building those traditional, vernacular houses is not being transferred. Almost all young people from villages are moving to cities in search for a better living. As soon as they are successful, they return to their respective villages and tend to build masonry houses as a show off of their success.

Batak – 1975



Lombok – 1977

Sumbawa – 1977

Minangkabau – 1977

Sukabumi – 1979

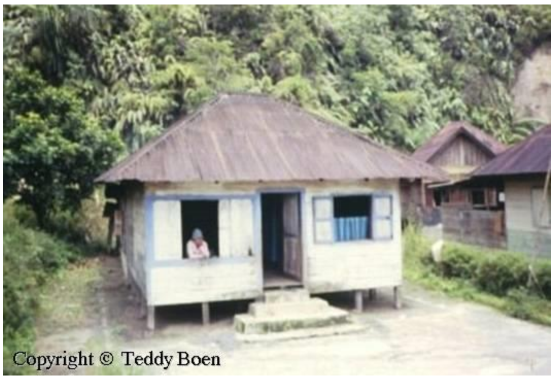
Tasikmalaya – 1979

Figure 13 – Traditional / Indigenous Buildings

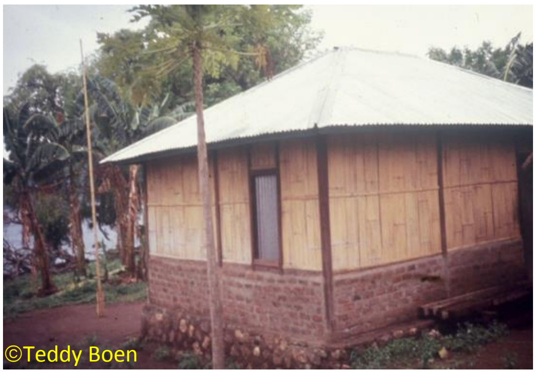
Manado – 1980

Flores (Lahayong) – 1982

Aceh – 1983



Tarutung – 1987



Flores – 1992

Halmahera – 1994

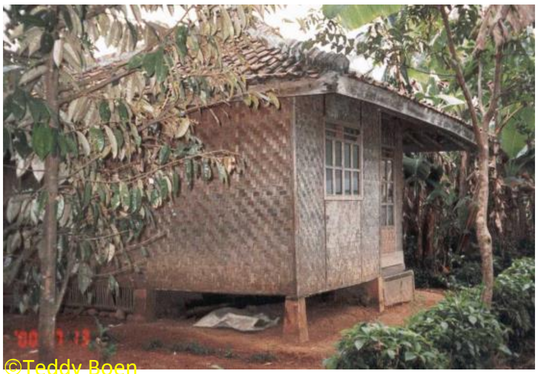
Serui – 1994

Liwa – 1994

Figure 13 (cont'd) – Traditional / Indigenous Buildings

Palu – 1995

Biak – 1996



Bengkulu – 2000

Pandegelang – 2000

Aceh – 2004

Nias – 2005

Simeulue – 2005

Bengkulu – 2007

Figure 13 (cont'd) – Traditional / Indigenous Buildings

3.2.2. Masonry Buildings

Masonry wall buildings will include load bearing masonry wall buildings, stud wall and brick nogged constructions in timber, and composite constructions using combinations of load bearing walls and piers in masonry, reinforced concrete, timber, and the like. The buildings which do not follow the requirements for masonry construction and which are built with poor workmanship and poor quality of materials, have very poor performance during past earthquakes and have taken a high toll of human lives and caused great property losses throughout the world.

Masonry buildings were introduced by the Dutch when Indonesia was a colony of the Dutch hundreds of years ago. This type of masonry buildings is copied from Europe and consists of one brick thick walls, using brick pilasters without any reinforced concrete columns and beams as confinement. At that time the Dutch used mortar mix consisting of burnt brick powder, lime powder and sand, mixed with water. Some used pozolan and lime mix as mortar. The strength of this type of mortar mix can be maintained provided that certain moisture content is maintained. During the Dutch occupation all such buildings were annually white washed with lime mixed with water. Such layer of paint is porous and during rainy season, rain water / moisture can penetrate and will be absorbed by the mortar, therefore the moisture content was maintained. Since the moisture content is maintained, the strength of the mortar is retained. However, with the introduction of new building materials, particularly in the past 40 years, including the introduction of acrylic, weather shield paints, most of the houses are painted with acrylic based paints. Acrylic seals the masonry wall surface and rain water can hardly penetrate. Therefore the moisture content in the mortar is not maintained and this makes the mortar very brittle. Thus the masonry wall becomes brittle and easily disintegrated when shaken by an earthquake. However, from the damage survey, in actuality, many of the masonry buildings following the Dutch tradition but built in the colonial era can still be found all over Indonesia, particularly in Pariaman, Bukitinggi, West Sumatra, in Yogya and Mid Java, used sand and lime only as mortar. This is apparently a common practice. The strength of lime and sand mortar is less than if mixed with burnt brick powder and this is also one of the causes of brittle failure. The foundation of most of this category buildings are river stone foundation without reinforced concrete foundation beams. Roof trusses are usually embedded in walls without proper anchoring (Boen, 2006).

The Dutch also applied plaster to any masonry surfaces with the same mortar as that for the joints. There are questions whether or not good bond can be obtained between the masonry wall surface and the plaster. If plaster is applied to new masonry construction surface, there is usually a good bonding surface available. Good bond is achieved because clay brick masonry walls and concrete blocks masonry walls usually have texture rough enough to provide good mechanical key and proper suction. Most of the surfaces texture of the masonry walls is coarse and rough for the plaster to stick to the surface and it has a good mechanical key. Good bond between masonry surfaces and the plaster can be achieved if there is a mechanical key and suction of water. Usually prior to plastering, the masonry wall surface is wetted with clean water so that the moisture will be soaked in to the masonry. If the water is absorbed by the masonry walls, it is a good indication that the plastering will stick to the masonry wall surface, thus get a good bond. In some cases, in hot weather, If the masonry wall absorbs too much water, the plaster quickly stiffened and become difficult to work properly This usually occurred during hot weather and suction must be controlled by spraying (not soaking) the masonry wall surface. If plastering is done long after the wall is finished, it is advisable to remove deteriorated and loose portions before applying the plaster (Portland Cement Association, 1975).

Most of the one or one and half-brick-thick masonry buildings built in accordance to Dutch tradition are very old and many are dilapidated due to lack of maintenance. As can be observed in many other earthquake disasters, there is a relationship between the age of the buildings and their quality particularly in non-engineered constructions. The deterioration of the materials, in particular the mortar, contributed to many of the damages and / or collapses of such one- or one-and-half-brick-thick masonry buildings. It was also observed that the lack of integrity between the various components, the foundation, the walls and the roof. Inadequate connections are causing the building to tear apart. Proper connections and detailing must be developed to improve the structural integrity of the non-engineered masonry buildings of this category. Non-engineered constructions in this category consist of houses, small shops as well as religious and one story school buildings (Boen, 2006).

Lombok – 1977

Yogya – 2006

Yogya – 2006

Padang – 2009

Figure 14 – One-brick Thick Masonry Buildings

After Indonesia becomes an independent nation, the demand for masonry buildings / houses is substantial and due to the increase in cost, people started building half-brick masonry houses. In the very beginning, those half-brick masonry buildings / houses were built without any reinforcement, so called Unreinforced Masonry (URM). Trass lime blocks, reinforced concrete hollow blocks was introduced in the 60’s. URM is quite brittle and would have difficulty withstanding the whiplash effect of an earthquake. Therefore, minimum reinforcement should be provided.

From surveying, studying and documenting some 49 earthquakes damages in various areas in Indonesia over the past 40 years, it can be stated that in almost all rural as well as urban areas all over Indonesia, a good earthquake resistant design feature can be identified, namely almost all half-brick-thick masonry buildings are built with reinforced concrete framing, consisting of the so called “practical columns and beams”.

According to Badan Pusat Statistik (BPS), from census 2010 in Indonesia, there are approximately 30,218,454 households in urban and 30,887,004 households in rural area. Figure 15 shows the approximate house in urban and rural areas for each province in Indonesia (see Appendix B). Almost 90% of the buildings in the earthquake stricken areas are masonry “non-engineered” buildings consisting of half-brick-thick confined masonry walls (Badan Pusat Statistik Republik Indonesia, 2012). The confinement consist of reinforced concrete framing, consisting of the so called “practical columns and beams”. “Practical columns”, size 120x120mm with four 10 or 12 mm diameter bars as longitudinal reinforcement and 8 mm stirrups spaced at 150-200 mm, are commonly cast after the construction of the masonry walls is complete, and sometimes the “practical columns” were cast first. “Practical beams”, size 150x200 mm with four 10 or 12 mm diameter bars as longitudinal reinforcement and 8mm stirrups spaced at 150-200 mm, are cast directly on top of the foundation and served as tie beams. Similar beams, size 120x200 mm with four 10 or 12 mm diameter bars as longitudinal reinforcement and 8 mm stirrups spaced at 150-200mm, are cast directly on top of the brick wall and served as ring beams (Figure 16). Almost all buildings have timber roof trusses with galvanized iron sheets roofing. Few buildings used clay tiles for roofing. The buildings mostly used saddle type roof trusses (Boen, 2006; Boen, 2007; Boen, 2007; Boen, 2003).

Typical concrete compression strengths range from 75-125 kg/cm^2 (JICA Manado Survey Team, 2009; JICA - Jurusan Teknik Sipil Universitas Negeri Padang, 2010; JICA - Aneka Asia Buana, PT, 2012) with rebar having a yield capacity of minimum 2400 kg/cm^2. The masonry infill wall is made of 50 x 100 x 200 mm burnt clay brick using running bond with mortar thickness ranging from 8-15mm. In the past 25 years, Portland Cement (PC) is used extensively and the mortar mix usually consists of 1cement : 3sand to 1cement : 4sand. The walls are plastered on both sides with sand and cement mortar of approximately 10 to 15 mm thickness. Such type of masonry construction has become a new culture all over Indonesia and from past earthquakes it is evident that provided they are built with good quality materials and good workmanship, they can survive the most probable strongest earthquake for 500 years return period in accordance with the Indonesian seismic hazard map (Boen, 2006; Boen, 2007; Boen, 2007; Boen, 2003; Center for Disaster Mitigation Institute Technology Bandung, 2011). The same masonry construction was analyzed using the 2012 Indonesian Seismic Map with the same return period can still survive.

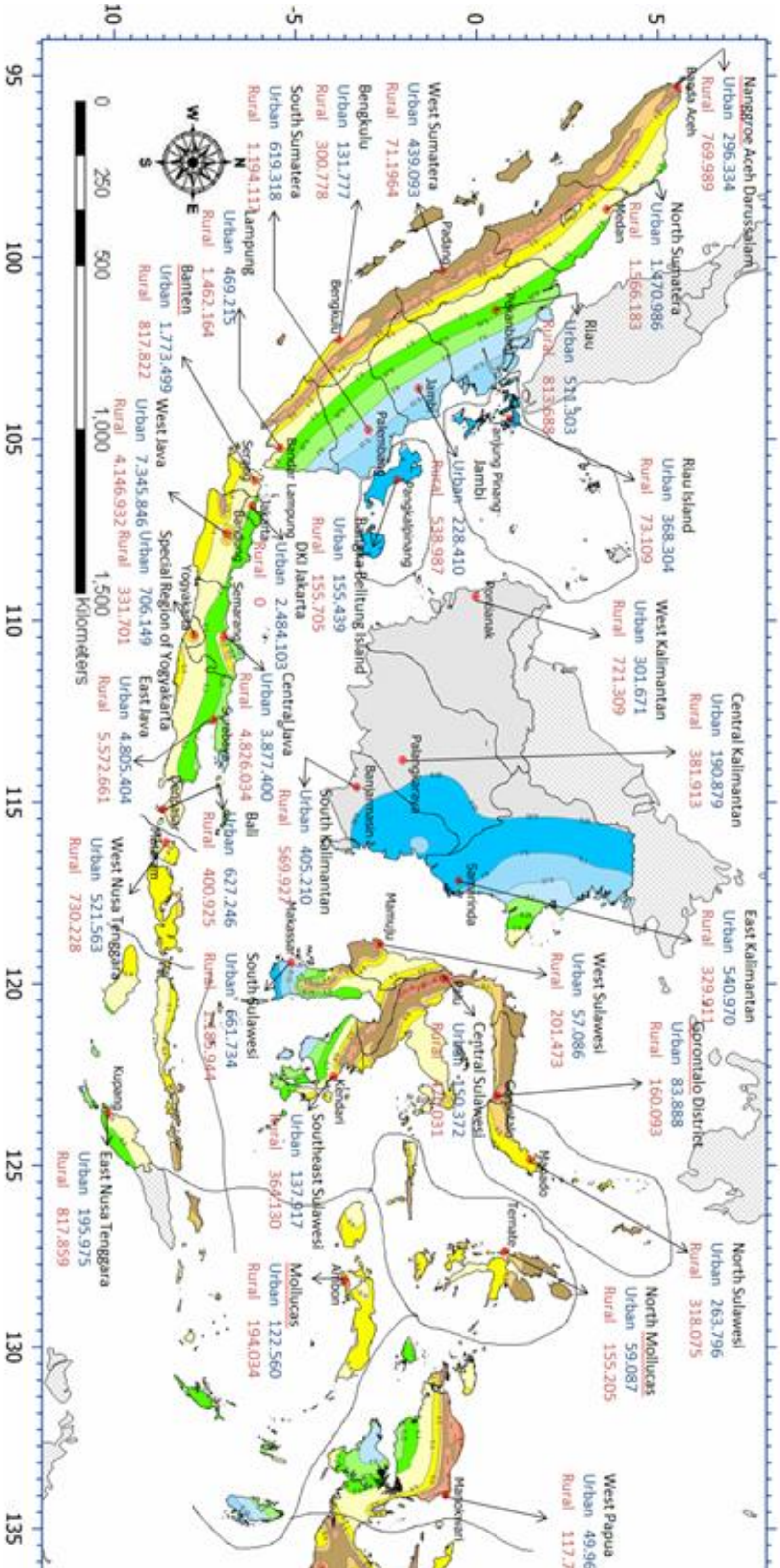


Figure 15 – Number of Houses in Urban and Rural areas in Indonesia

Note: numbers of houses are based on Badan Pusat Statistik Republik Indonesia, 2012.

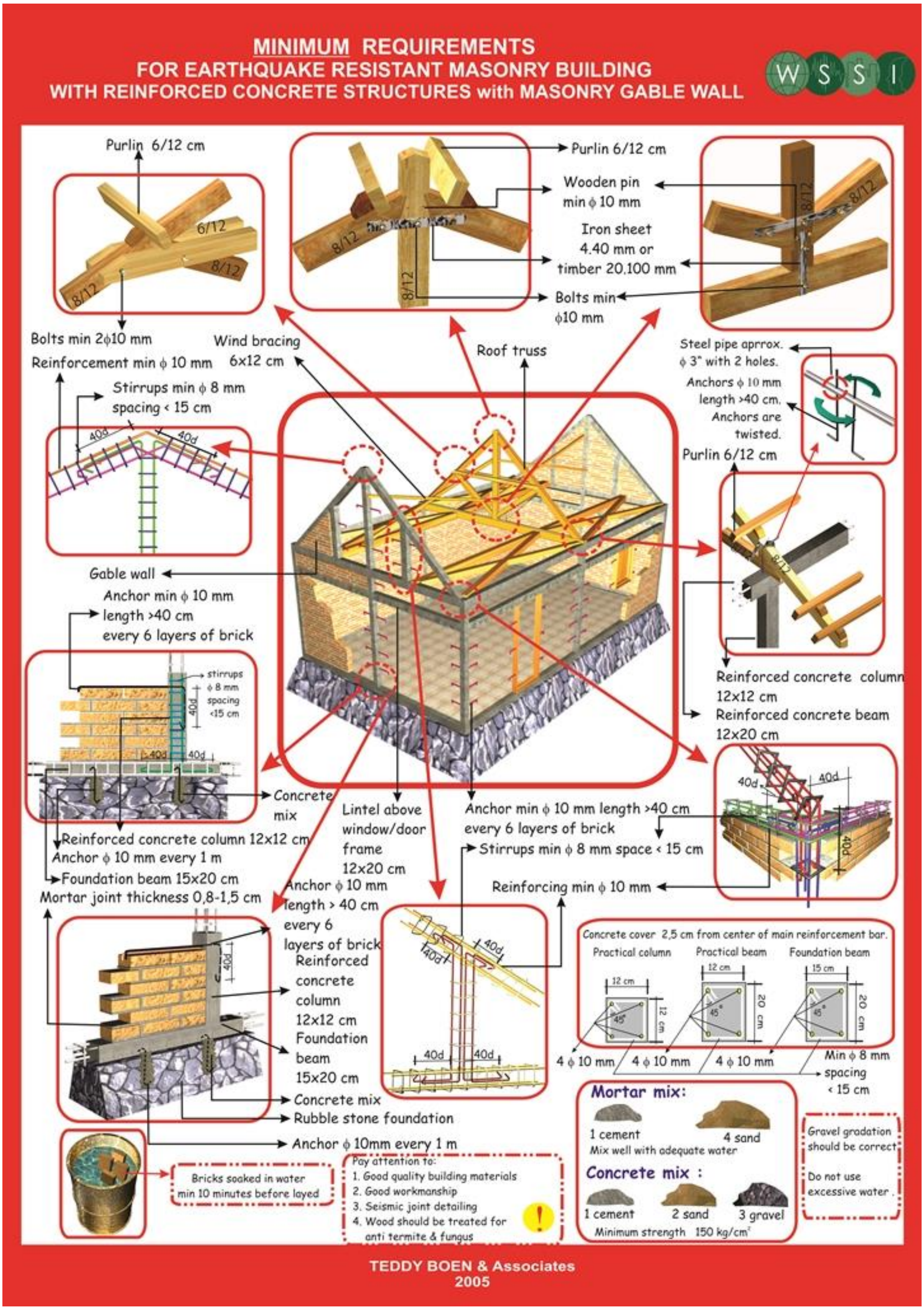
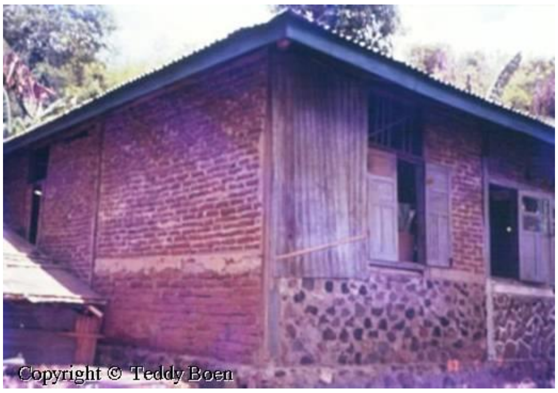


Figure 16 – Poster Minimum Requirements for Earthquake Resistant Masonry Buildings with Reinforced Concrete Structures

Sukabumi – 1979

Tasikmalaya – 1979

Manado – 1980



Flores (Lahayong) – 1982

Serui – 1994

Liwa – 1994

Kerinci – 1995

Palu – 1995

Figure 17 – Half-Brick-Thick Confined Masonry Buildings

Biak – 1996

Bengkulu – 2000

Padang – 2007

Padang – 2007

Figure 17 (cont'd) – Half-Brick-Thick Confined Masonry Buildings

Besides using reinforced concrete "practical columns" and "practical beams", half-brick-thick masonry buildings also can be confined using timber.

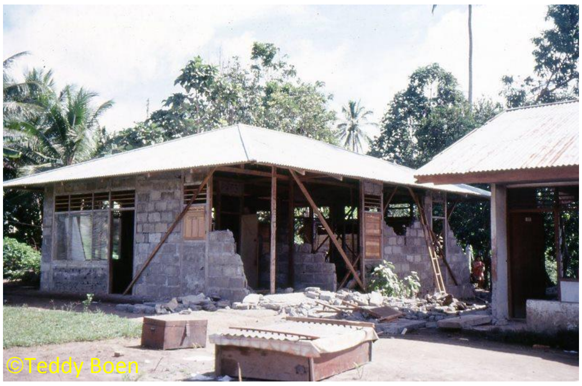


Figure 18 – Masonry Buildings Confined with Timber Columns and Beams

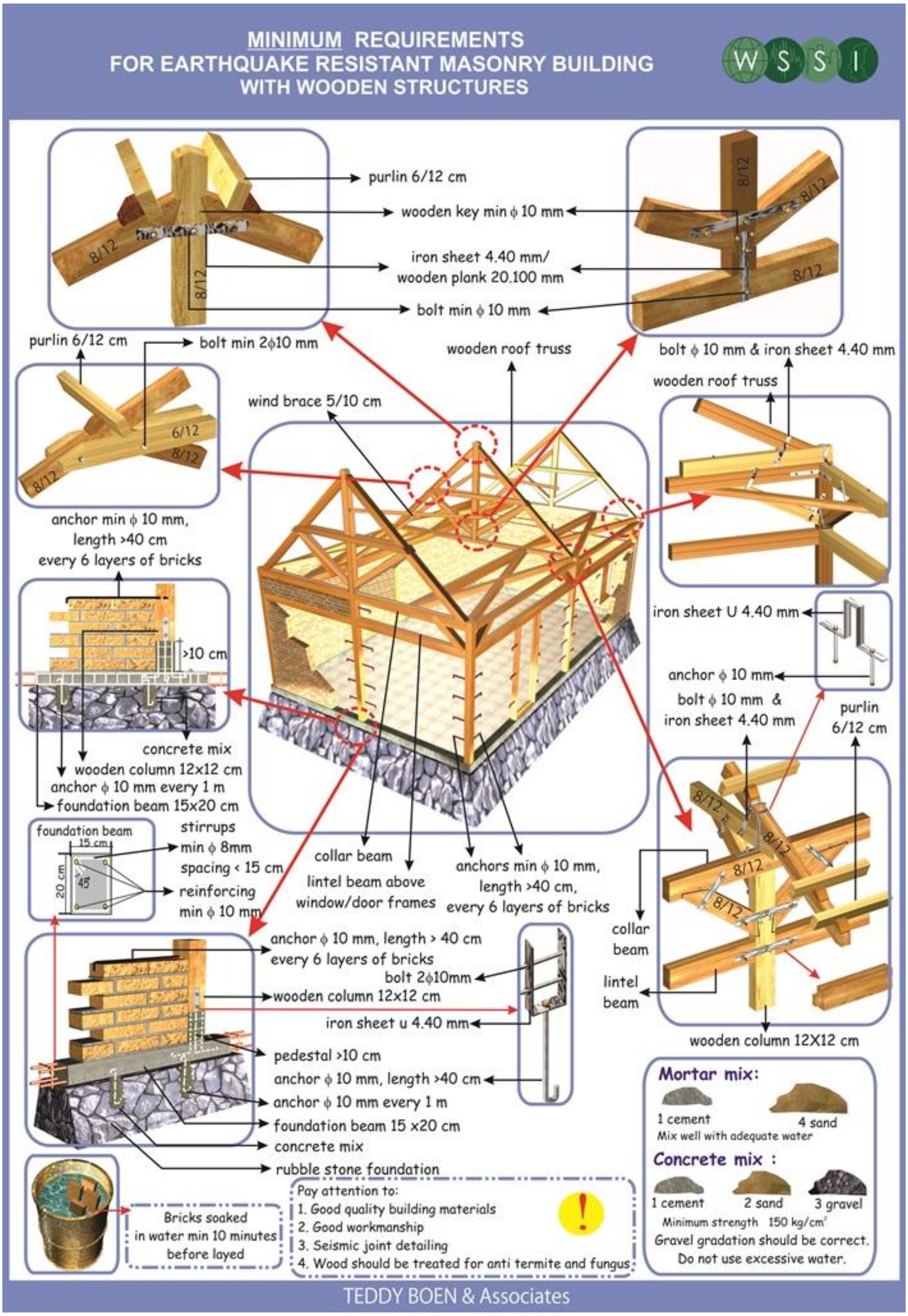


Figure 19 – Poster Minimum Requirements for Earthquake Resistant Masonry Buildings Confined with Timber Columns and Beams

From survey of earthquake damages in the past 40 years, the author also found that in several places in Indonesia, masonry buildings are confined using reinforced concrete "practical columns and beams" with bamboo as reinforcement. Bamboo has a good characteristic to withstand tensile forces; therefore bamboo can be used as a substitute of reinforcing bars, if only the bamboo's shrinkage can be reduced approximately equal to the concrete's shrinkage. Bamboo also has a weakness and difficult to bend like steel bars and therefor the column - beam joint is not ductile. From past earthquake experiences, one of the failure modes of masonry buildings is caused by improper connection at joint of columns and beams.

Figure 20 – Masonry Buildings Confined by "Practical Columns and Beams" with Bamboo as the Reinforcement

In general, non-engineered construction in Indonesia can be summarized as described in Table 2. Most of the buildings in the earthquake stricken areas are masonry non-engineered constructions consisting of half-brick-thick masonry walls. This type of buildings is earthquake resistant if built based on earthquake resistant principles. However, from past 40-year surveys of significant damaging earthquakes in Indonesia, many masonry non-engineered constructions were damaged and/or collapsed during earthquakes. This fact proves that the masonry non-engineered constructions are not earthquake resistant and there are some problems in non-engineered constructions in Indonesia that may cause many victims and economy loss at every earthquake as mentioned before in Chapter 2.

Table 2 – Summary of Non-Engineered Construction in Indonesia

Non-Engineered Construction	Built in	Performance
1. Traditional / Indigenous Buildings	before the colonial era	– based on local traditional wisdom – used materials available in that area – have a good record or good performance in past earthquakes
2. Masonry Buildings • One-brick masonry	Dutch occupation	– using brick pilasters without any reinforced concrete columns & beams – mortar mix consisting of burnt brick powder, lime powder & sand; or only used sand & lime – dilapidated due to lack of maintenance
• Half-brick masonry ○ Unreinforced masonry walls ○ Reinforced masonry walls	after independence "new culture"	– without reinforcement – very brittle; not earthquake resistant – using reinforced concrete or timber "practical columns" & "practical beams" – earthquake resistant if built based on earthquake resistant principles

3.2.2.1. School Buildings in Indonesia as Non-Engineered Construction

According to the Ministry of Education and Culture, in 2012, there are 148,167 school buildings for Elementary School and 36,381 school buildings for Junior-High School in Indonesia. Figure 21 shows the number of school buildings for each province in Indonesia (see Appendix C). Most school buildings in Indonesia, majority were built in 1970's and 1980's, can be considered as non-engineered constructions. Most school buildings are single storied (more than 85 %), few two storied (about 10 %) and just about 5 % are three storied. The story height is generally 3m for primary schools and 3.0-3.50m for others. The classroom blocks usually consist of single row of rooms with a narrow covered veranda in front, the blocks being connected through a passage of light open construction.

The common building materials used for school buildings in Indonesia are wood, burnt clay bricks, reinforced concrete, and cement mortar. Wood is very commonly used for structural as well as non-structural purposes. Most school buildings have sloping roofs made of timber roof trusses, rafters and purlins and carrying galvanized iron or asbestos sheet roofing. The most common structural type for one story school buildings is a wall bearing construction. Where load bearing walls are used, there is a wooden or reinforced concrete wall plate on which the trusses rest and are anchored to it through nails.

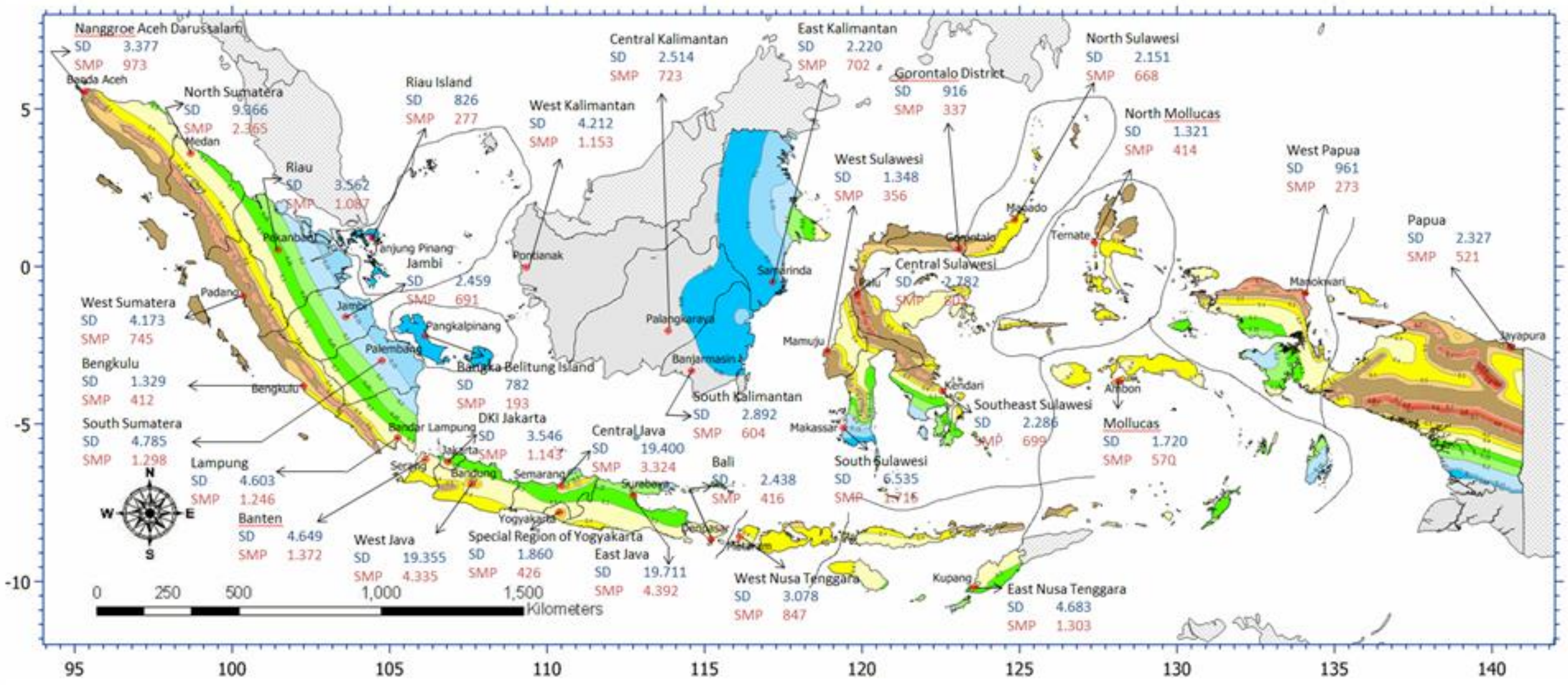


Figure 21 – Number of School Buildings in Indonesia

Note:

- SD (Sekolah Dasar) = Elementary School
- SMP (Sekolah Menengah Pertama) = Junior-High School
- Numbers of school buildings are based on http://www.sekolahdasar.net/2012/10/jumlah-sd-di-indonesia-ada-148361.html

The walls are commonly constructed of burnt clay bricks (having sizes 5cm x 10cm x 20 cm and 6 cm x 12 cm x 22 cm) with a wall thickness of 15 cm. Some very old building walls are in random rubble 40 cm thick. The mortar in rural areas is cement and sand ranging from 1: 5 to 1:10 ratio. New construction, particularly in cities use cement–sand mortar in 1: 3 to 1:5 ratio and walls are 15-16 cm thick with or without any frame. Alternatively reinforced concrete "practical" columns (size 15 x 20 cm and size 12 x 12 cm) are used under the trusses and brick walls are used as tight infill. In such cases a reinforced concrete foundation ring beam and collar beam (size 15 x 20 cm) at roof level are used. Many school buildings using timber posts and timber ring beams and trusses with plywood/ or timber planks side covering and GI roofing have also been adopted as light construction. In very remote areas, bamboo matting is also used as wall cladding in rural areas. For single storied school buildings, shallow river stone footings are used for bearing and non-bearing walls and isolated footing for columns.

From the above explanation, the methodology of building one- and two-story school buildings is similar to non-engineered masonry construction.

3.3. Characteristics of Masonry Buildings

A masonry wall is a composite structure made of masonry units and mortars. In Indonesia, masonry units can be from burnt clay bricks (bricks) or concrete blocks (solid or hollow). Lately, light weight concrete blocks are used, particularly in high-rise buildings. The basic mortar used during the Dutch occupation was lime sand mortar (LM), lime burnt-clay brick powder and sand mortar (CLM), and since approximately 25 years ago when Portland Cement (PC) was produced in large quantity in Indonesia, cement sand mortar (CM) is widely used, therefore, the "quality" of mortar is slightly improved.

The most important characteristic of masonry construction is its simplicity (Mosalam, et al., 2009). Laying pieces of stone or bricks on top of each other, either with or without cohesion via mortar, is a simple, though adequate, technique that has been successful ever since long time ago. Other important characteristics are the aesthetics, solidity, durability, low maintenance, versatility, sound absorption and fire protection.

Masonry is strong in compression and weak in tension (O'Brien & Dixon, 1995). Under compressive loading, the strength of the wall is influenced by the strength of both materials, among others, the strength and the shape of the masonry units, the composition and thickness of the mortar joint, and the bond between the mortar and the unit. To exploit the structural potential of any material, it is essential to understand its strengths and its weaknesses (Curtin, et al., 1987).

In masonry wall using good quality concrete blocks and sand-cement mortar, the elastic modulus of the mortar is usually substantially less than that of the blocks. This condition is opposite if the masonry wall using burnt clay bricks with sand-cement mortar; the elastic modulus of mortar is larger than bricks.

There are two extreme possibilities interaction of block and mortar strength (Roberts, et al., 1983):

- Between horizontal joints, all loads will effectively be carried by the blocks while at the horizontal joints, all the load is carried by the mortar so that the wall strength might be expected to correspond to the strength of the weaker material
- The function of the mortar joint is simply to produce good uniform bearing between the blocks and provided the mortar is not so fluid that it could squeeze out like tooth paste, its strength is irrelevant and the wall strength will correspond to the strength of the blocks. In fact, the second possibility is closest to the truth though the properties of the mortar may have some influence on the strength.

The interaction between block and mortar strength above is also applicable for bricks.

3.4. Constructing Masonry Wall Buildings in Indonesia

The common practice in Indonesia for masonry walls are half-brick-thick masonry walls using confinement in the form of foundation beams, practical columns and ring beams (see Figure 22). Past earthquakes showed that such type of houses are earthquake resistant provided that they are built properly using good quality materials, good workmanship and all building components (foundation, columns, beams, walls, roof trusses, roofing) must be tied each other, so that when shaken by earthquakes, the building will act as one integral unit (Boen, 2005). Masonry non-engineered constructions are usually moderate in size and consist of many walls. If all the walls are appropriately connected to each other, such buildings will act as a rigid box-like structure and can withstand when shaken by earthquakes.

In order to build earthquake resistant masonry buildings, there are some issues that should be considered:

- <u>Diaphragms (walls)</u>: in elastic design of masonry buildings, the diaphragm must be a rather stiff element. The purpose of the diaphragm is to distribute the loads from the out-of-plane walls to the in-plane walls through the diaphragm itself.
- <u>Connections</u>: critical importance in the seismic design of all buildings. Connections that join perpendicular walls shall be continuous across the structure wherever possible. The fundamental properties of a well-designed masonry connection are:
    - Must be durable under cyclic loading
    - Does not cause major damage to surrounding bricks/blocks, (local failure serves to dissipate energy).
    - Attachment or bearing on a masonry surface must be distributed over as large an extended bearing surface as possible.
- <u>Structural continuity</u>: provided by elements that can transfer dynamic forces between structural elements in either the vertical or horizontal direction. The continuous transfer of loads between parallel walls does much to increase the stability of the walls.
- <u>Compatibility</u>: compatible elements are those that have similar load – deflection characteristics. When incompatible elements are combined in the same system, large forces can be generated between the incompatible members as the system deforms. Cracks regularly form and propagate at the location of material incompatibilities in masonry walls (e.g. door and window frames).

- Structural redundancy: redundancy refers to the availability of alternative paths for resolution of earthquake induced forces in a structure. When there is only one path, the loss of that path can have catastrophic impacts on the structure. When there are several, then the loss of one need not have major consequences.

Earthquake resistant one- or two-story school buildings use the same principles (Figure 23).

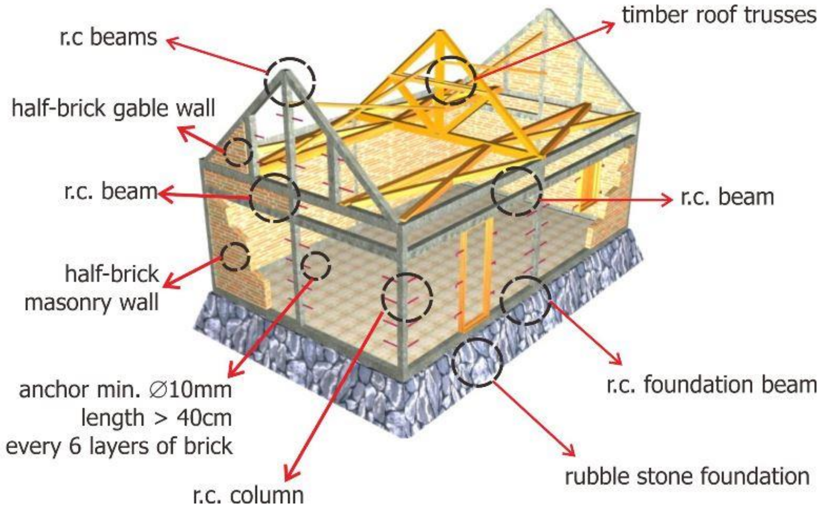


Figure 22 – Earthquake Resistant Confined Masonry Building

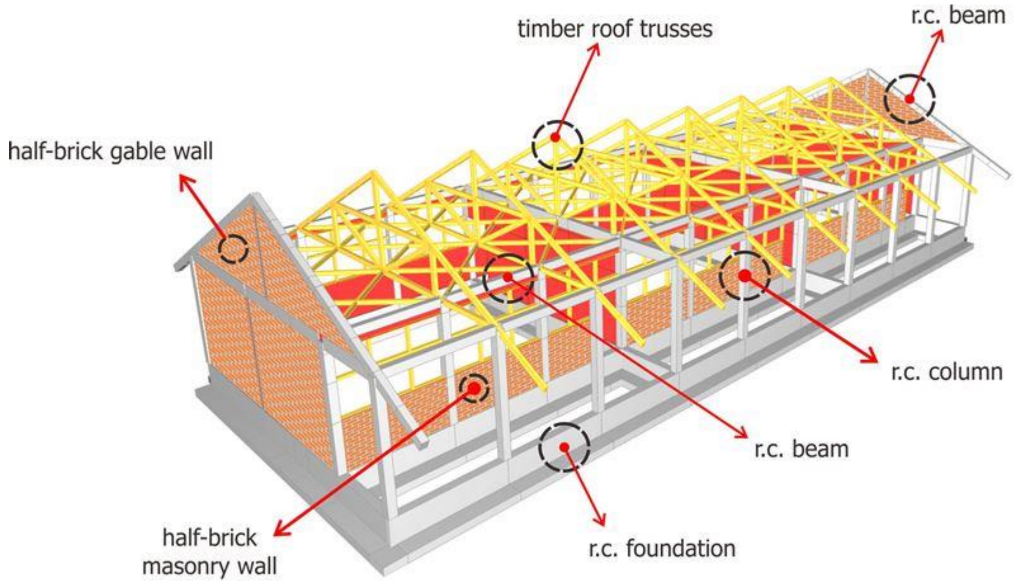


Figure 23 – Construction of One-Story School Buildings

Below are the sequences to construct seismic resistant masonry houses (Boen, 2005):

Erection of Batter Boards

Batter board is used as benchmark for the levels of the house. It shall be erected prior to construction. Usually the batter board dimension is 2x20cm, supported by timber stakes 5x7cm and placed every 2m apart (see Figure 24). The upper part of batter board must be flat, smooth, and horizontal. At corners, the batter boards must be perpendicular to each other.

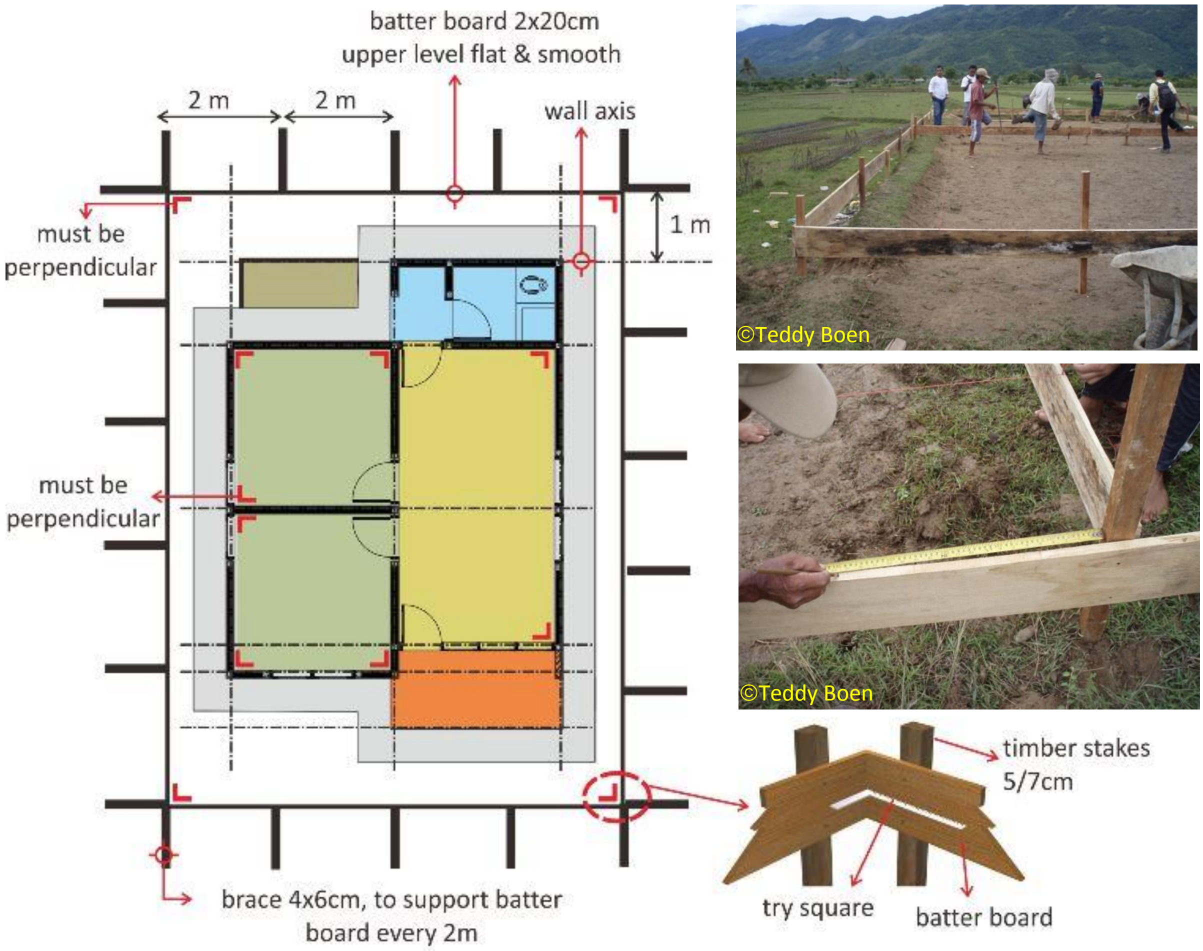


Figure 24 – Erection of Batter Board

Reinforcement Preparation

In conjunction with the preparation of the site, the reinforcement of beams and columns must also be prepared (Figure 25).

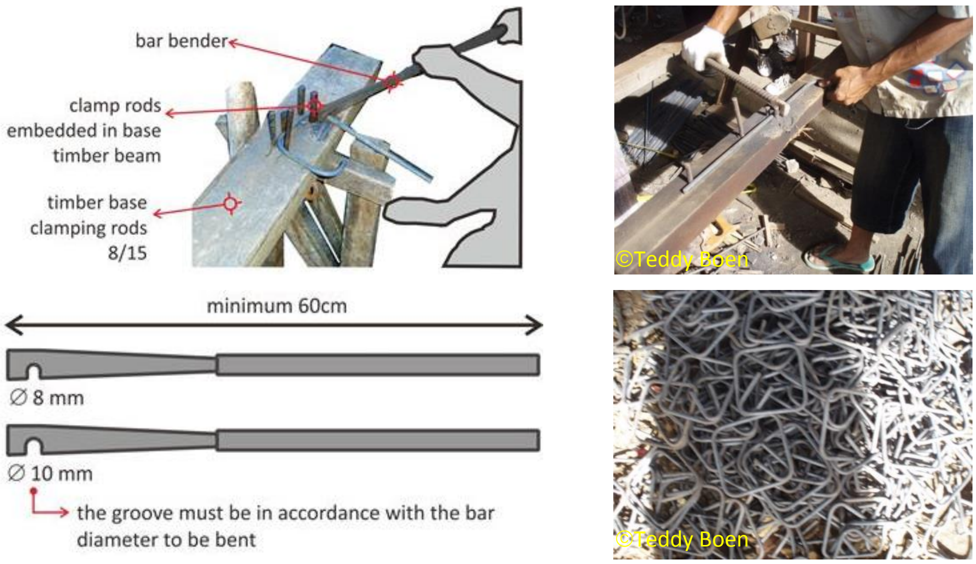


Figure 25 – Preparation of Beam and Column Reinforcing Bars

Example: length calculation of column reinforcing bars

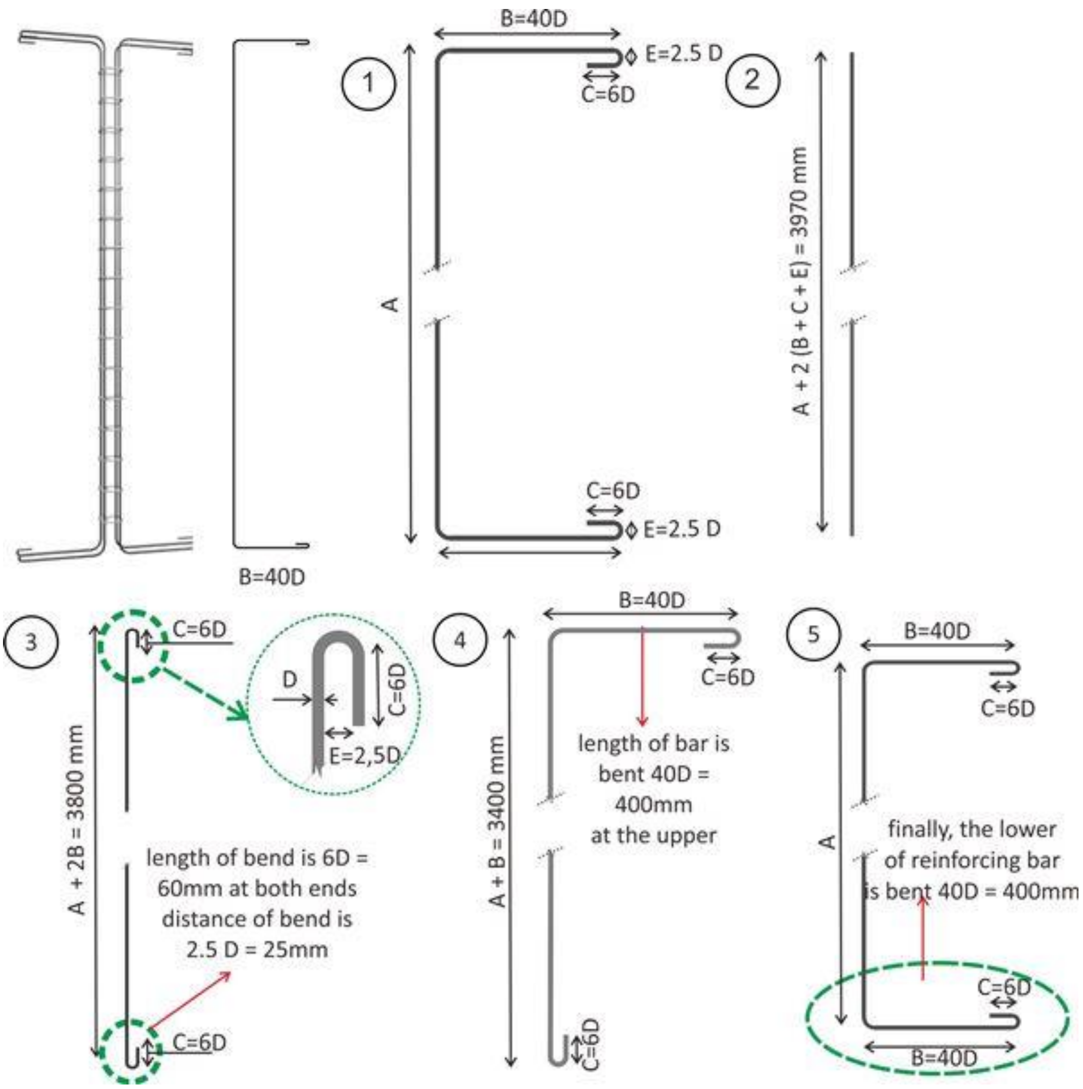


Figure 26 – Example Length Calculation of Column Reinforcing Bars

Column with 3m height from axis to axis, using bar ∅10mm:

Formula: A + 2 (B + C + E)

- A = 3000mm
- B = 40D = 400mm
- C = 6D = 60mm
- E = 2.5D = 25mm
- D = bar diameter = 10mm

Length of column reinforcing bar = A + 2 (B + C + E) = 3000 + 2 (400 + 60 + 25) = 3970mm

Therefore, for 12m reinforcing bar, it can be obtained 3 column reinforcing bars for 3m height from axis to axis.

Prior to cutting reinforcing bars, the lengths of columns, beams reinforcing bars and stirrups, and length of hooks must be determined from construction drawings. After the reinforcing bars are cut based on the necessary length, the reinforcing bars are bent with appropriate bar bending tool and shaped into columns, beams, stirrups (Figure 27). Bending bars after the reinforcing bars are assembled is not correct (Figure 28).

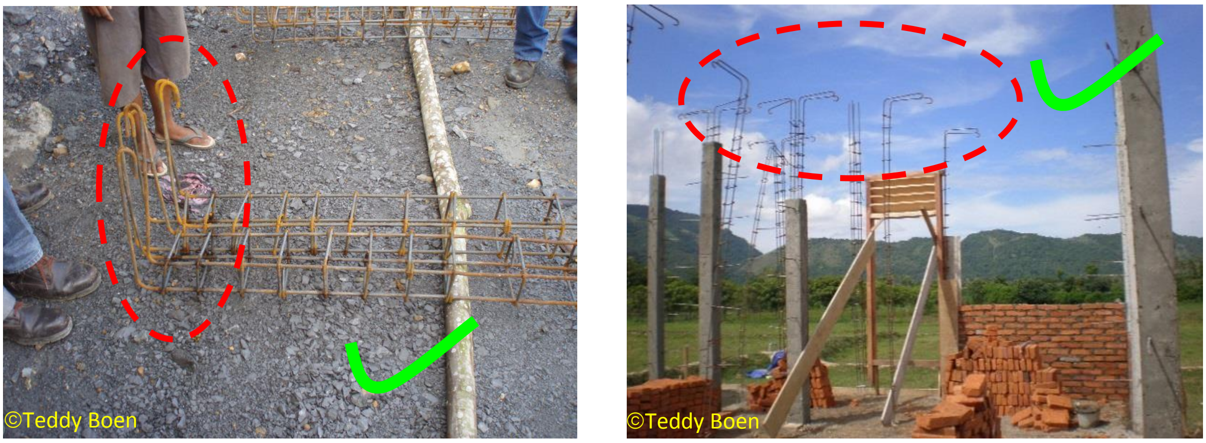


Figure 27 – Assembling of Column Reinforcing Bars

Figure 28 – Incorrect Assembling of Column Reinforcing Bars – DO NOT Follow the Principles of Seismic Resistant

Foundations Excavation

After all batter boards are constructed, the foundation excavation is done (Figure 29). The minimum depth and width is 80cm, and depends on the foundation dimension.

Figure 29 – Foundation Excavation

Constructing Foundations

Foundations are built using rubble stone (Figure 30). Such rubble stone foundation was introduced by the Dutch. However, since several years ago, the author suggested to use continuous reinforced concrete inverted-T foundation (Figure 31).

If the foundations are using continuous reinforced concrete inverted-T type, the foundation beams has integrated with the foundation. Therefore the column reinforcing bars should be assembled together with the foundation reinforcing bars.

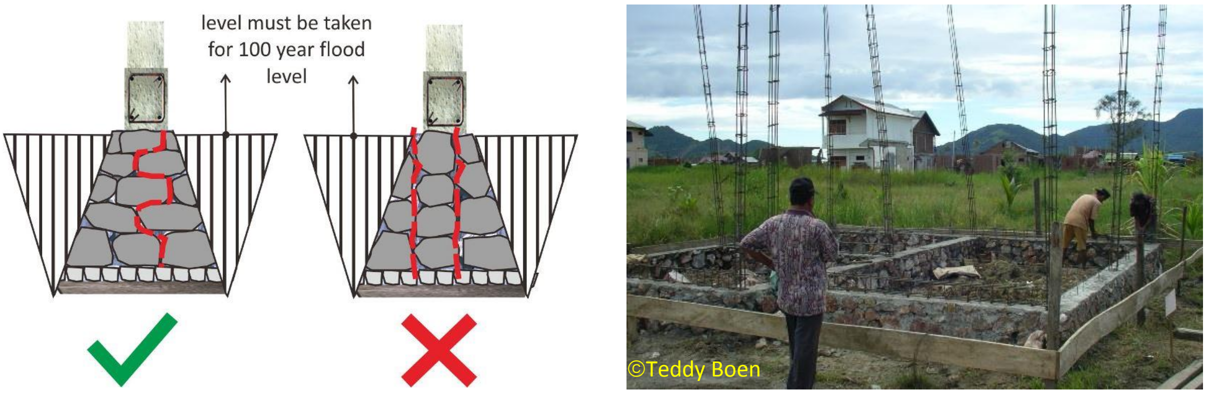


Figure 30 – Rubble Stone Foundation

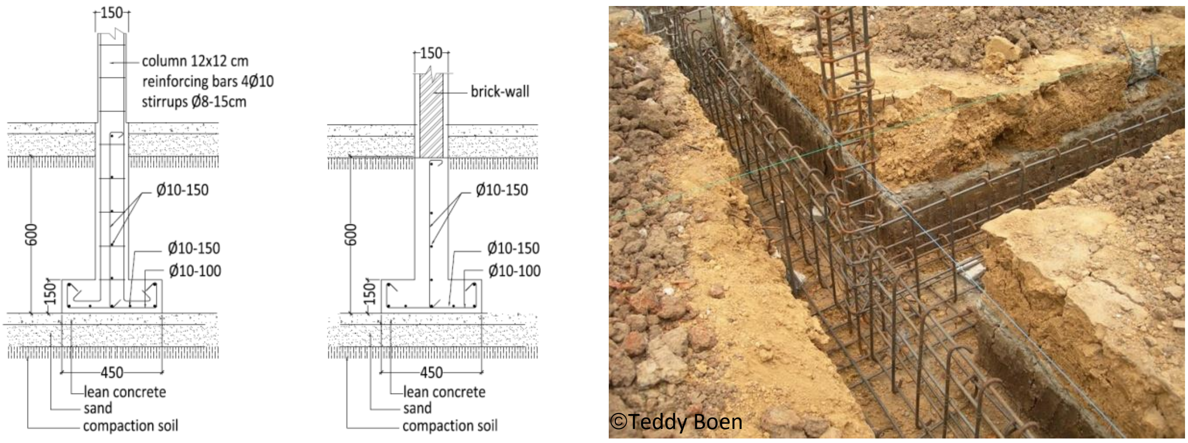


Figure 31 – Reinforced Concrete Inverted-T Foundation

Preparing Concrete Mix

The ratio of water : cement : sand : gravel that meets standard requirement is 1:2:4:6 or ½:1:2:3. Therefore material requirements for mix 1 m^3 of concrete are 0.125m^3 water, 0.250m^3 cement, 0.500 m^3 sand, and 0.750 m^3 gravel. The expected minimum compressive strength is approximately 150 kg/cm^2. To test the concrete mix consistency, place the concrete in your hand. The appropriate concrete mix will be achieved if the concrete can be grasped by your hand (Figure 33 – left). However, if the water is too much, the concrete mix will be "melted" in your hand (Figure 33 – right).

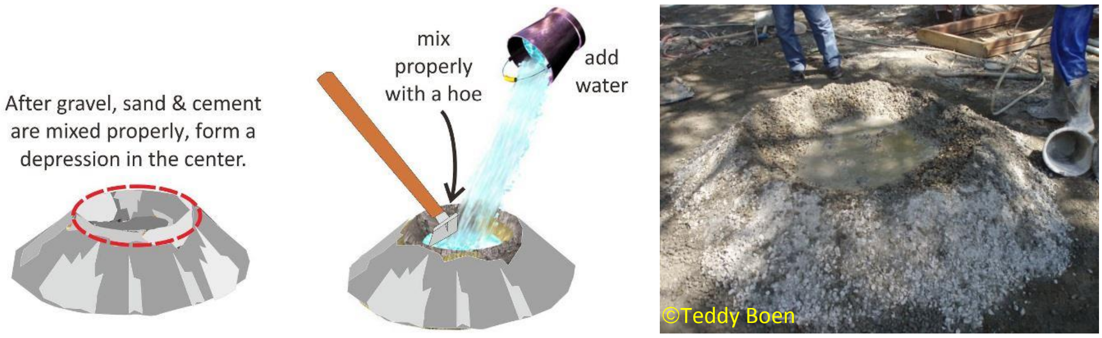


Figure 32 – Preparing Concrete Mix

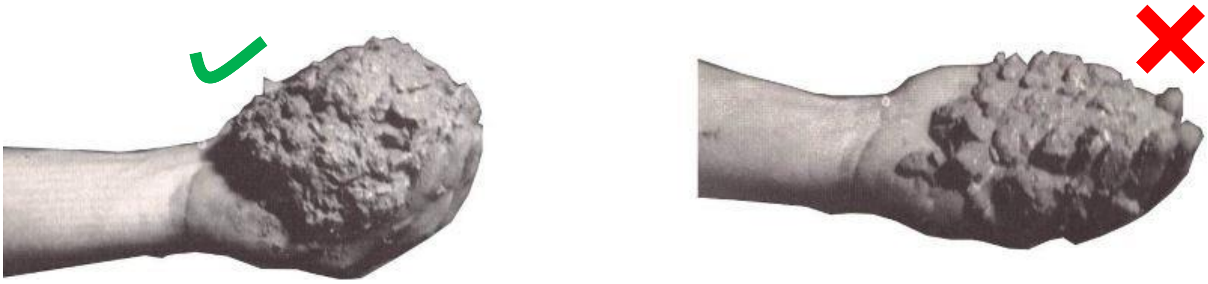


Figure 33 – Concrete Mix Consistency Test

Constructing Foundation Beams

The column reinforcing bars should be assembled together with the foundation beams reinforcing bars to ensure an appropriate seismic detailing. Detailing of the reinforcement must be in accordance with Figure 34.

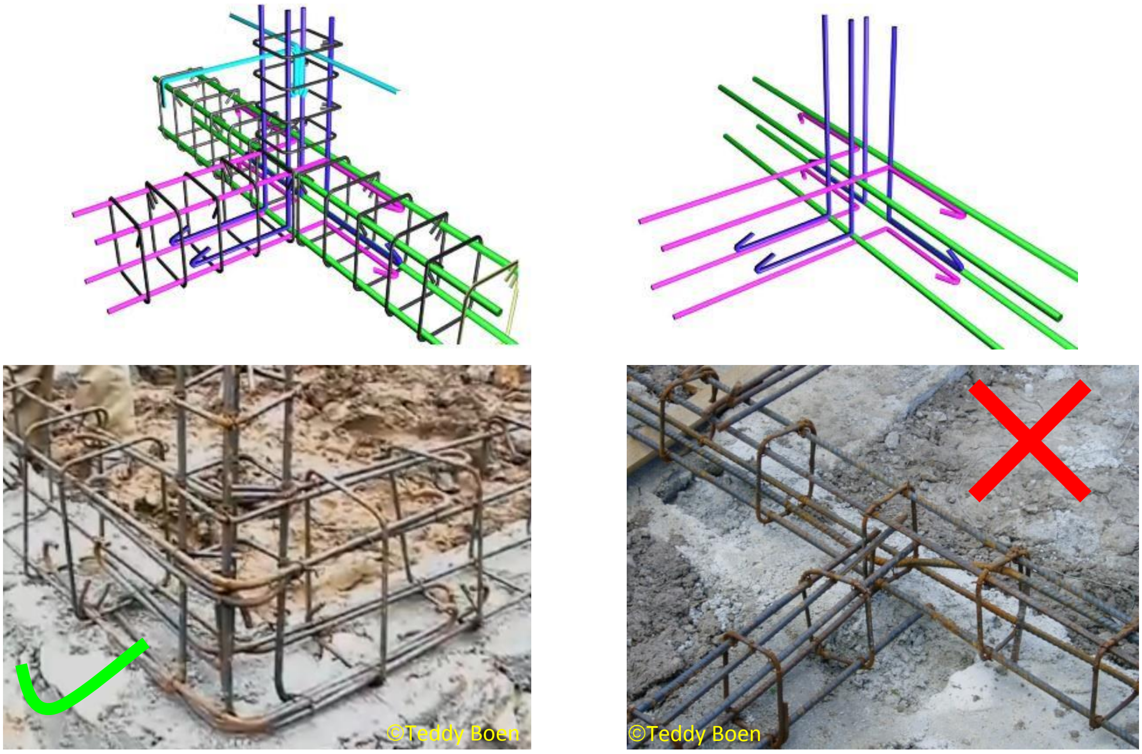


Figure 34 – Foundation Beam Reinforcing Detailing

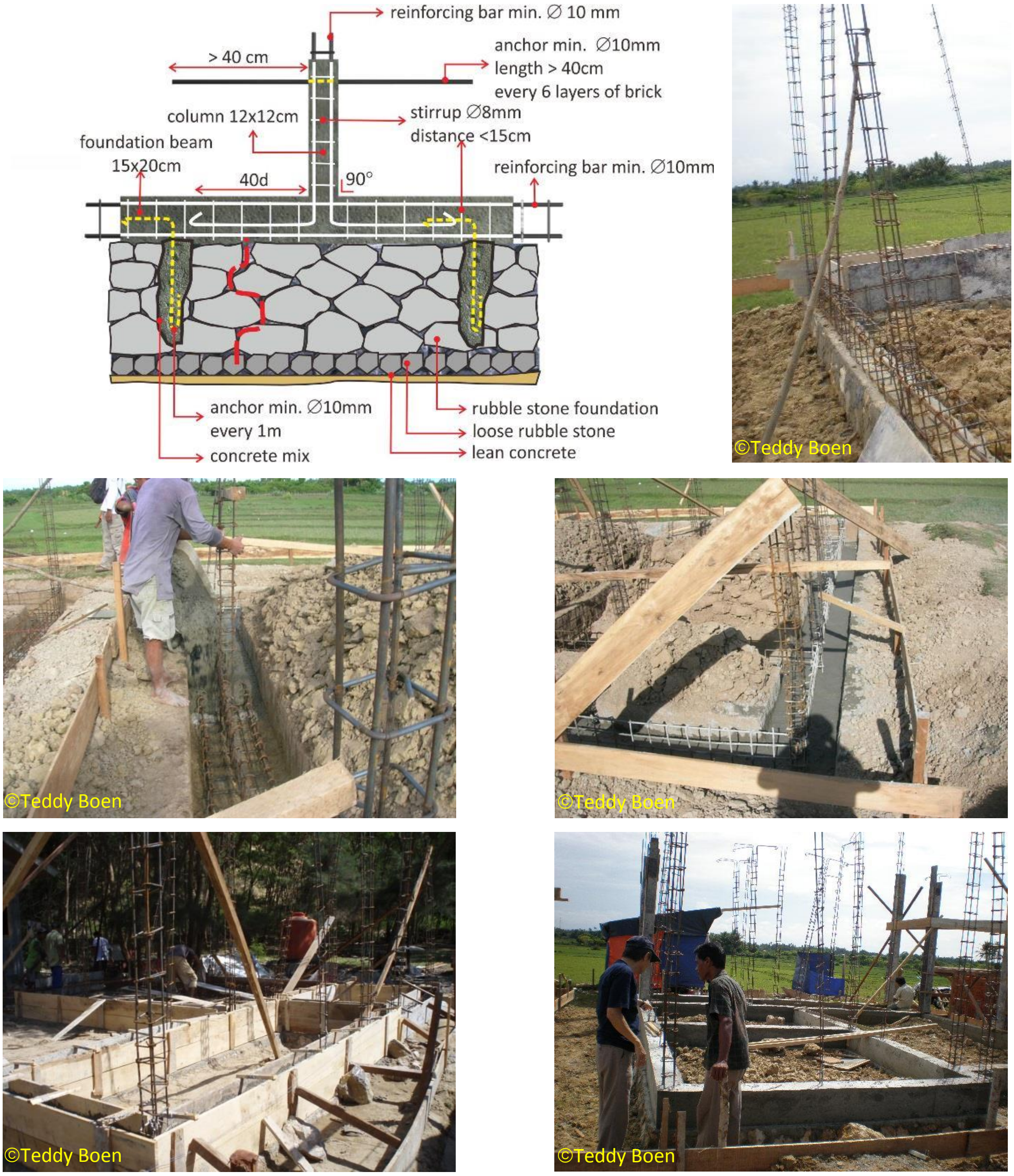


Figure 35 – Foundation Beams Construction

Curing of all reinforced concrete components must be done before and after the form work is removed. It must be sprayed routinely to prevent evaporation of concrete mix water. Lack of water in concrete may cause fissures that can reduce the concrete strength.

Figure 36 – Curing of Concrete

Constructing Columns

The reinforcing bar of columns shall be assembled coincide with the foundation reinforcing bars. However the concreting of columns can be implemented in two ways. The first is simultaneously with brick laying (Figure 37). The second is prior to brick laying (Figure 38). The columns are supported on four sides to warrant plumbness during placing of concrete. Wall anchors must be assembled before concreting columns. Placing concrete is done in one run and NOT IN STAGES.

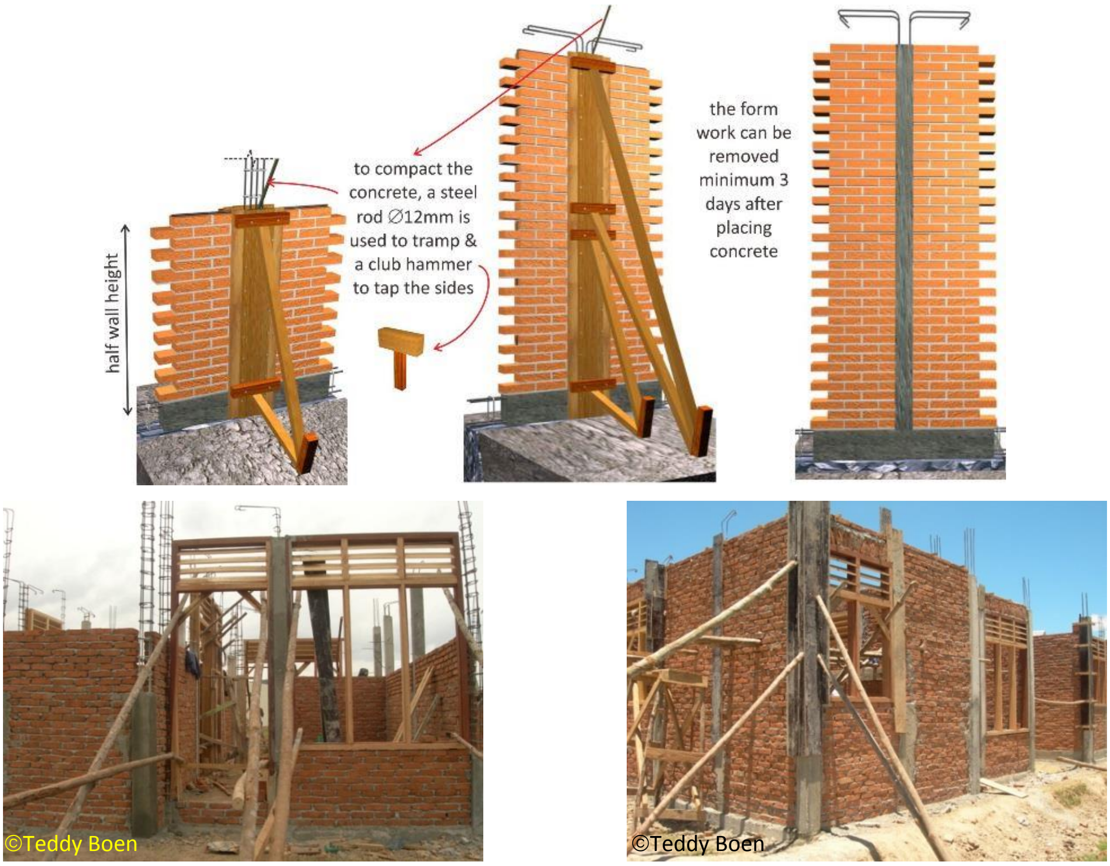


Figure 37 – Concreting Columns Simultaneously with Brick Laying

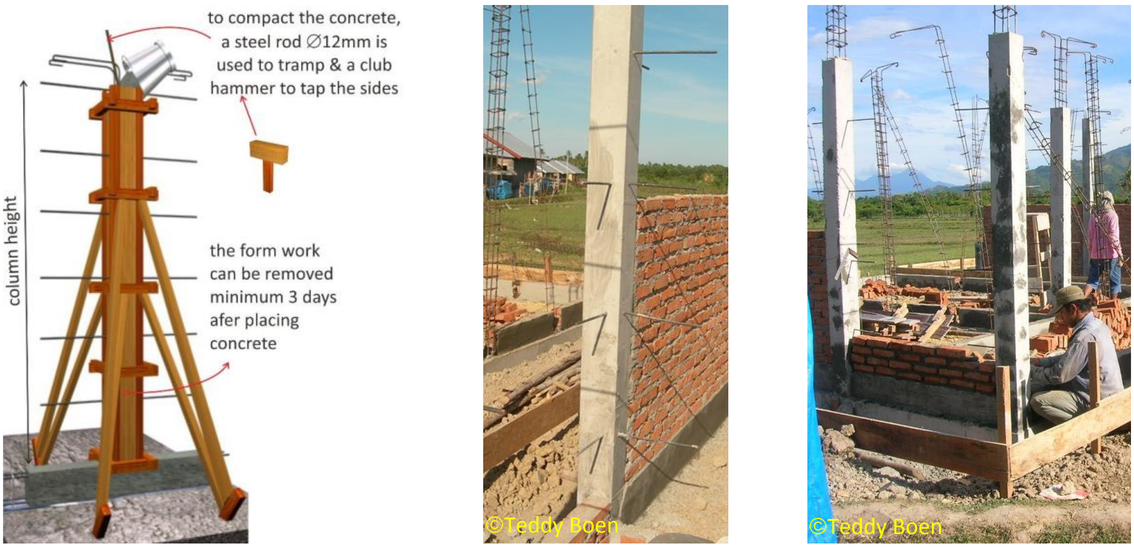


Figure 38 – Concreting Columns Prior to Brick Laying

Constructing Masonry Brick-Walls

Burnt clay bricks are usually used to construct the walls. Simple method to check the quality of the bricks is shown in Figure 39. The brick-walls must be anchored to the columns using minimum ∅10mm reinforcing bars with length minimum 40 cm, every 6 layers of brick. The bricks must be soaked in water minimum 10 minutes prior to laying and shall be laid immediately to prevent evaporation (Figure 40). When laying the bricks, a cord is used as a horizontal guideline (Figure 41). The ratio of cement and sand in mortar is 1 pc : 4 ps.

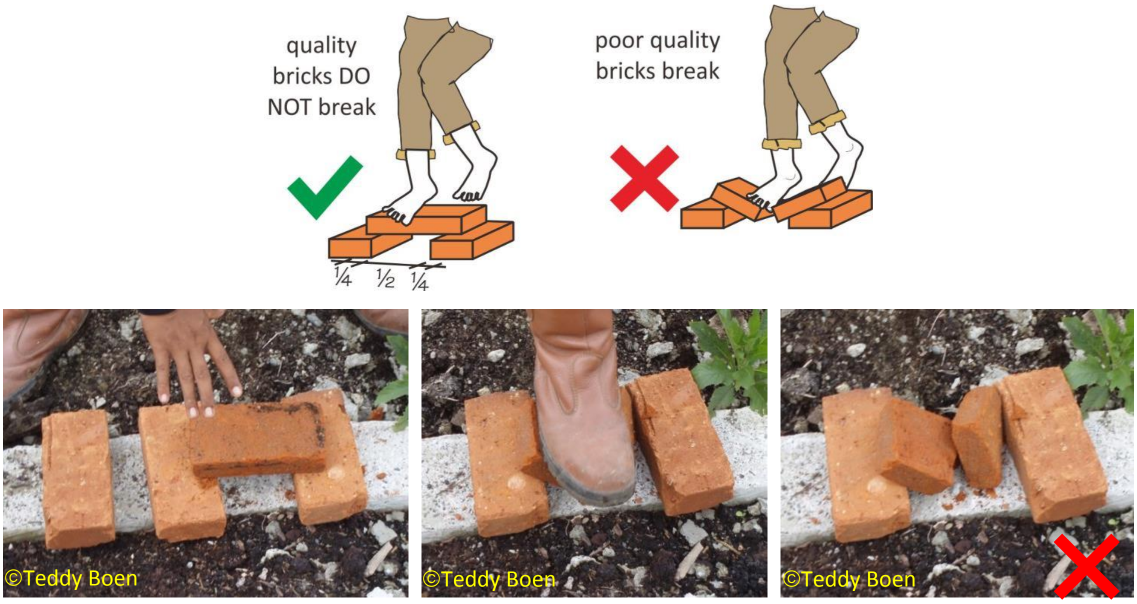


Figure 39 – Quality of Brick Testing

Figure 40 – Brick Soaked in Water

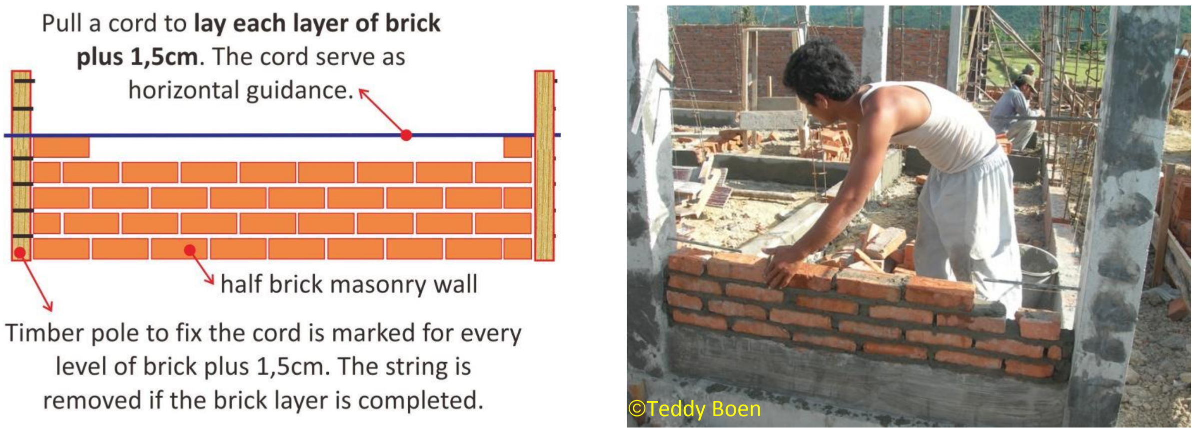


Figure 41 – Brick Laying

Constructing Ring Beams

After the construction of columns and walls are completed, the ring beams are constructed. The reinforcing bars for ring beams shall be in accordance with seismic detailing requirement as shown in Figure 42.

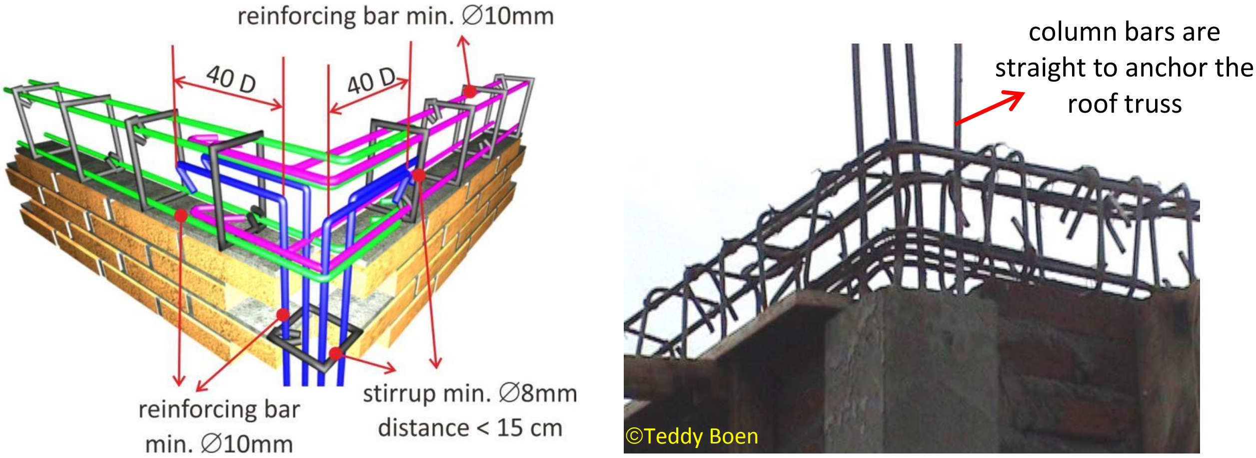


Figure 42 – Ring Beams Reinforcing Bars Assembling

Erection of Roof-Trusses

Roof trusses shall be anchored to ring beams (see Figure 43).

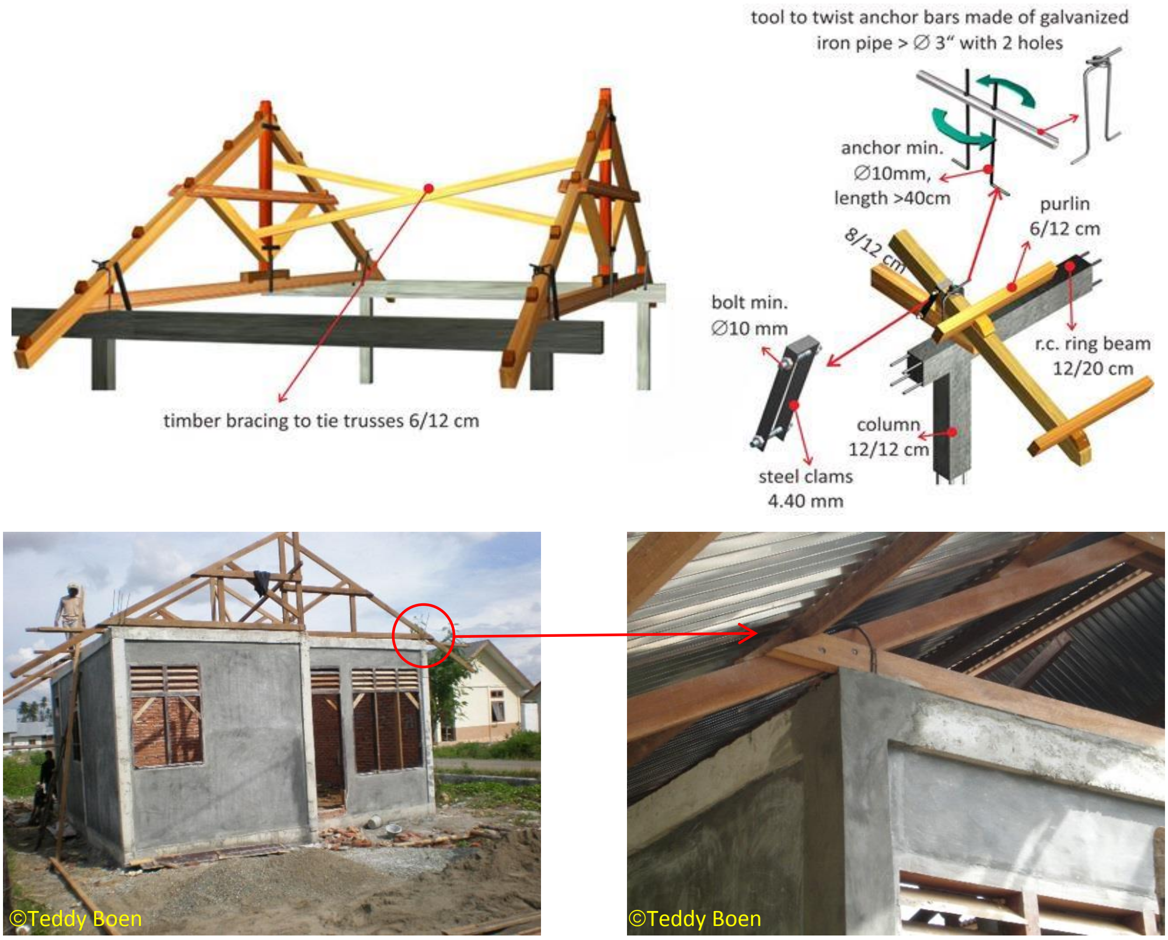


Figure 43 – Erection of Roof Trusses

3.5. Experiments of Non-Engineered Masonry Construction

3.5.1. Confined Brick-Wall Panels

The experiment of confined brick-wall panels was performed in PUSKIM, Bandung, sponsored by JICA (JICA - Aneka Asia Buana, PT, 2012; Research Institute for Human Settlements & JICA, 2012). There were 13 variant of brick-wall panels tested which varies in column size, reinforcement, mortar strength, brick quality and concrete quality. However this dissertation will only illustrated two major brick-wall panels with significant differences:

1. The first brick wall panel with column dimension 15x15cm with longitudinal reinforcement 4∅10mm and stirrup ∅8-150mm; concrete mix 1PC : 2 sand : 3 gravel : 0.8 water (average compressive strength 190.79kg/cm^2); mortar 1PC : 4 sand (average compressive strength 98.88kg/cm^2); and good quality of bricks (average compressive strength 58.0kg/cm^2).
2. The second brick wall panel with column dimension 10x10cm with longitudinal reinforcement 4∅8mm and stirrup ∅8-250mm; concrete mix 1PC : 2 sand : 3 gravel : 1.2 water (average compressive strength 162.83kg/cm^2); mortar 1PC : 7 sand (average compressive strength 73.71kg/cm^2); and low quality of bricks (average compressive strength 43.3kg/cm^2).

There were no plasters in both brick-wall panels. Each brick-wall panel has approximately 3.3m height and 3.0m width. The schematic confined brick-wall panels can be seen in Figure 44. The average rebar tensile strength for ∅8mm is 3400 kg/cm^2 with ultimate strength 5170 kg/cm^2 and for ∅10mm is 3325kg/cm^2 with ultimate strength 5188 kg/cm^2.

The test results provide information to create F-D diagram that is needed for non-linear analysis.

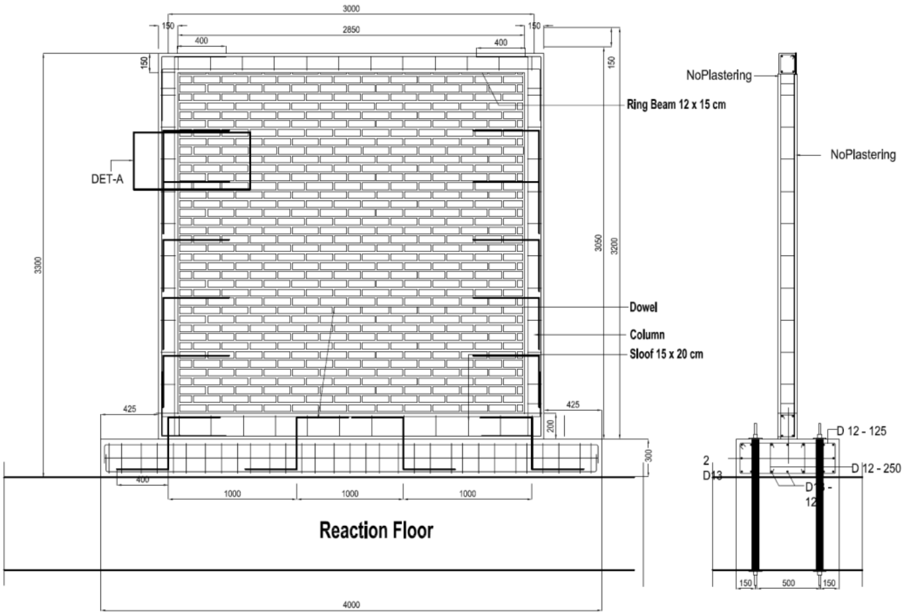


Figure 44 – Schematic of Confined Brick-Wall Panels Tested in PUSKIM (Research Institute for Human Settlements & JICA, 2012)

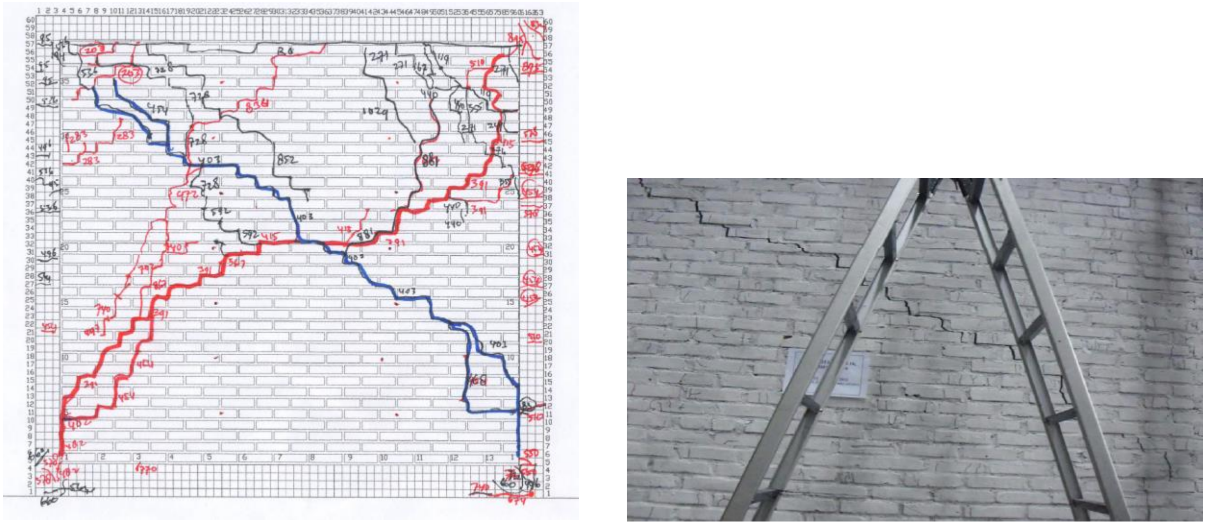


Figure 45 – Cracks Pattern of First Brick-Wall Panel with Good Quality of Bricks (Research Institute for Human Settlements & JICA, 2012)

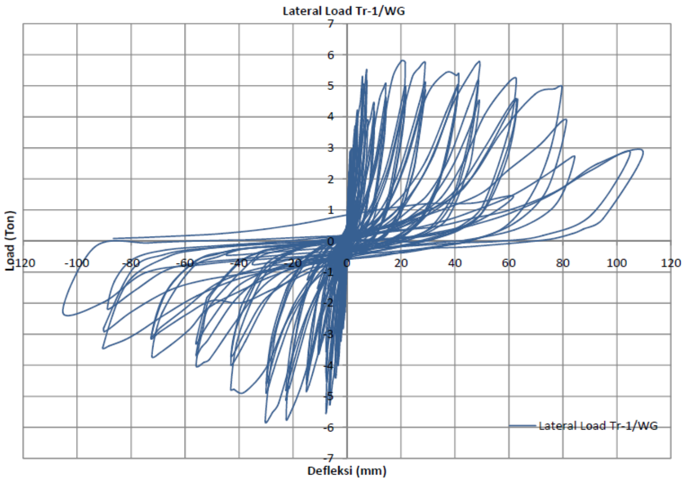


Figure 46 – Hysteresis loop of First Brick-Wall Panel with Good Quality of Bricks (Research Institute for Human Settlements & JICA, 2012)

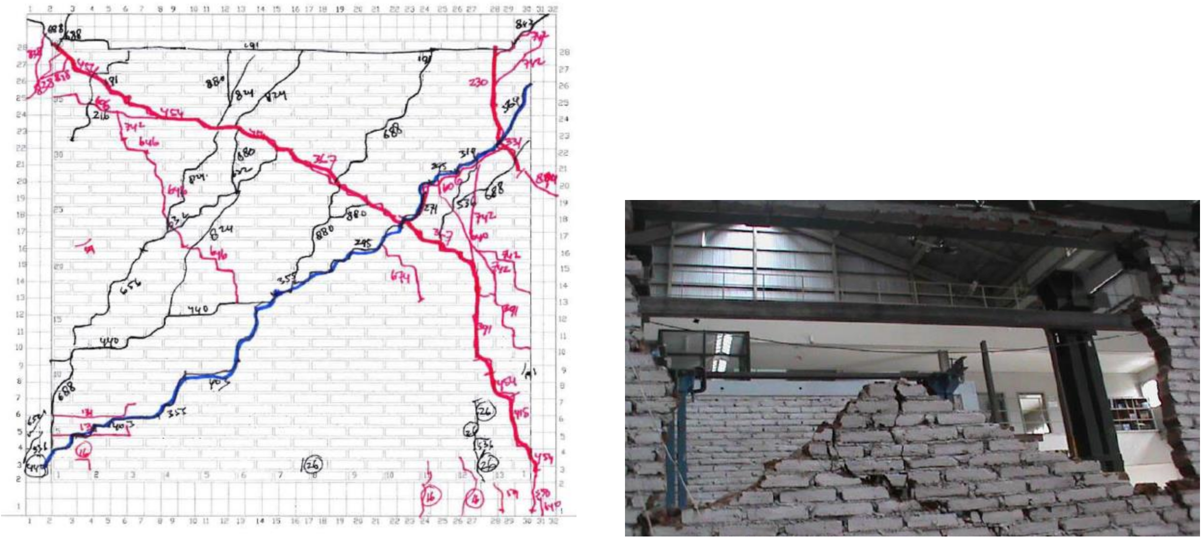


Figure 47 – Cracks Pattern of Second Brick-Wall Panel with Low Quality of Bricks (Research Institute for Human Settlements & JICA, 2012)

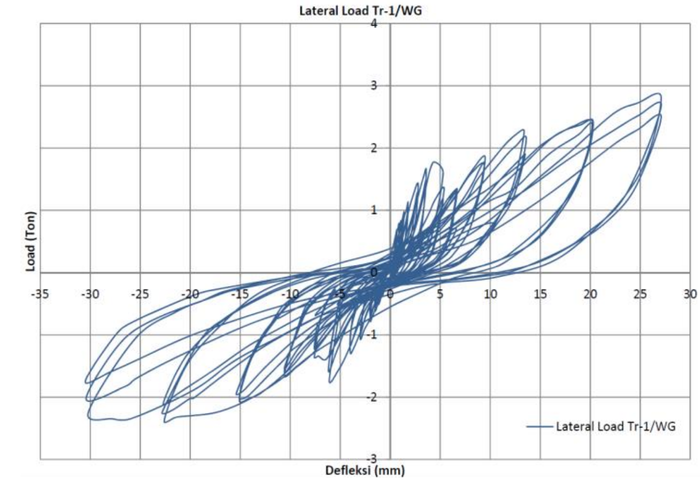


Figure 48 – Hysteresis loop of Second Brick-Wall Panel with Low Quality of Bricks (Research Institute for Human Settlements & JICA, 2012)

Below are the summaries of shaking table test results of masonry walls that have been conducted.

3.5.2. Shaking Table Test of One Brick Thick Masonry Walls Construction

On December 27, 2007 a shaking table test was performed in Tsukuba, Japan. The model was designed by Mie University and NWFP University of Peshawar, Pakistan. Although the model is based on one brick thick wall Pakistan houses with English bond, the result can be applied for Indonesia since one brick thick wall masonry houses also exists with similar material quality and workmanship (Minowa, et al., 2010).

The objective of the shaking table test is to study the actual seismic behavior of vulnerable masonry house.

3.5.2.1. Structure Model

The length, width and height of the model structure was approximately 3 meters, fabricated on the shaking table at the National research Institute for Earth science and Disaster prevention (NIED) in Tsukuba. The bricks used for the model structure were imported from Pakistan. The ratio of cement to sand for mortar is one to eight to take account of the vulnerability of rural houses in developing countries. The joint mortar thickness is approximately 15mm (Minowa, et al., 2010).

East wall, south wall and north wall have openings, but the west wall is without opening. The walls were built in accordance to English bond. Bricks of 230mm x 110mm x 70mm and 2.92kg were imported from Pakistan.

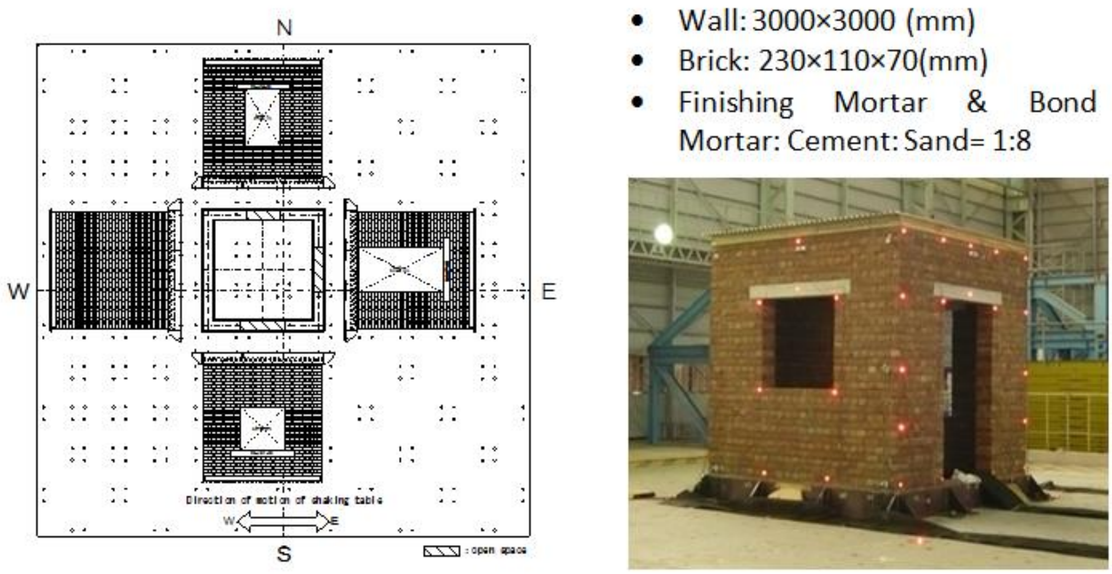


Figure 49 – Outline of Model Structure for Shaking Table Experiment of One Brick Thick Masonry Walls Construction (Narafu, 2010)

3.5.2.2. Material Properties

The average compressive strengths in three specimens of bricks were 147kg/cm^2 for bricks, and 92kg/cm^2 for cubic mortar test. The elasticity modulus was estimated 77,000 kg/cm^2 for brick, and 11,000kg/cm^2 for mortar in material tests. Lintels were installed above openings. Weight of the house model was about 10.23ton (bricks: 7.74ton, mortar: 1.79ton, lintels: 0.37ton, roof: 0.25ton) (Minowa, et al., 2010). Many houses in Indonesia built during the Dutch occupation are the same as the model tested; unconfined one brick thick masonry walls, therefore, the test results are also applicable for Indonesia.

3.5.2.3. Input Motions

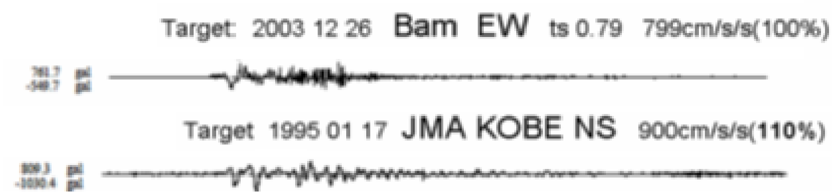


Figure 50 – Input Motion Records for Shaking Table Experiment of One Brick Thick (Narafu, 2010)

The shaking table was excited by sinusoidal waves, rectangular waves, and strong earthquake records. Two strong earthquake motions were used. First, component wave which was observed at Bam Governor's Building in Iran Earthquake on December 26, 2003; called as Bam. The second was a NS component wave which observed at JMA (Japan Meteorological Agency) Kobe Observatory in 1995 Hansin Great Disaster; called as JMA KOBE.

Three dimension image processing was carried out to know the dynamic performance of the brick walls. 4 high resolution cameras were used for measuring the dynamic response of the model structure.

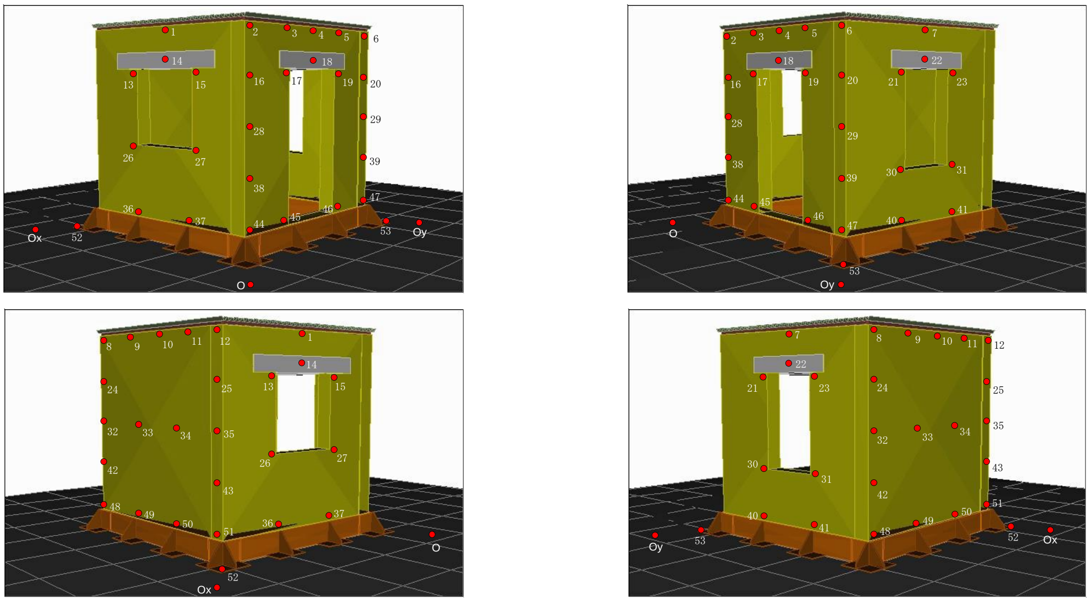


Figure 51 – The Area Watched by Each High Resolution Cameras for 3D Image Measurement (Narafu, 2010)

3.5.2.4. <u>Experiment Results</u>

Even though the model has been shaken by strong earthquake records: 2003 Bam L (EW) TS=0.79 100cm/s and 1995 JMA KOBE NS100cm/s (110%), there was no indication of damage at all. This indicates that brick masonry under controlled construction was rigid and strong to withstand to ever record strong earthquake motions.

However due to the objective of the shaking table test was to obtain the data on the collapse process of brick masonry structures in developing countries, the model must be tested until collapse on the shaking table test. Since no cracks occurred in the planned excitations, the excitations for making cracks in masonry walls were carried out additionally. The subsequent excitations to make the model collapse can be seen in Table 3.

Table 3 – Subsequent Excitations to Make the One Brick Thick Masonry Walls Construction Model Collapsed (Minowa, et al., 2010)

No.	Excitation	Result
1	2003 Iran Bam Eq. L (EW) TS=0.79 75cm/s	No damage
2	2003 Iran Bam Eq. L (EW) TS=0.79 100cm/s	No damage
3	1995JMA KOBE NS100cm/s (110%)	No damage
4	Sinusoidal 15Hz 1G 50second	No damage
5	Sinusoidal 1Hz 0.4G 20second	No damage
6	Pulse Shock 1 40cm/s	Cracks occurred

No.	Excitation	Result
7	Pulse Shock 2 -40cm/s	Cracks developed
8	Pulse Shock 3 30cm/s	No cracks development
9	2003 Iran Bam Eq. L (EW) TS=0.79 100cm/s	Cracks development
10	1995JMA KOBE NS100cm/s (110%)	Collapsed

In the shaking steps of No.6, No.7, No.8, velocity pulse shocks of 40cm/s and 1.7G was applied with amplitude 20mm, interval period 10s. Due to those shocks, distinct cracks appeared in walls. By crack appearance, the test of aiming collapse would be possible. After No.6, No.7, No.8, a Bam motion of No.9 was applied cracks became large. Accelerations of 1.5G–2G were measured at roof position. The deformation about 30mm – 50mm was observes by image processing. The last excitation, JMA KOBE motion of No.10 was applied and finally in less than 10s, the model collapsed. Cracks were similar to typical crack patterns of shear walls.

(2 + 15/30 sec) *(3 + 01/30 sec)* *(3 + 23/30 sec)*

(4 + 22/30 sec) *(5 + 16/30 sec)* *(6 + 17/30 sec)*

(6 + 26/30 sec) *(7 + 09/30 sec)* *(8 sec)*

Figure 52 – Sequence of Collapse of One Brick Thick Masonry Walls Construction due to 1995JMA KOBE NS100cm/s (110%) (Narafu, 2010)

3.5.3. Shaking Table Test of Confined Masonry Brick-Walls Construction

In order to provide the study for countermeasure against earthquake damages, the dynamic collapse test of a confined masonry house commonly used in Indonesia was conducted on July 4, 2008. National research Institute for Earth Science and Disaster Prevention (NIED), and MIE University conducted the dynamic failure test of confined masonry models by the use of NIED Tsukuba Shaking Table, in cooperation with Building Research Institute, Mitsuishi Fire Brick Co. Ltd and Tokyo Denki University (National Research Institute for Earth Science and Disaster Prevention (NIED), 2008; Minowa, et al., 2010).

3.5.3.1. Structure Model

The half-brick-thick confined masonry house of Indonesia was chosen. The house model has no roof with dimension approximately 3m each in length and height. East, south and west walls were made of Pakistan brick with dimension 230x110x70mm. Each walls of Pakistan brick wall had 32 layers. North wall was made of Japanese brick with dimension 210×100×60mm. The Japanese brick wall had 36 layers. Walls were built up with half bond masonry wall with ratio of cement to sand of mortar is one to eight. The model was based on the Indonesian practice for confined half-brick-thick masonry wall construction.

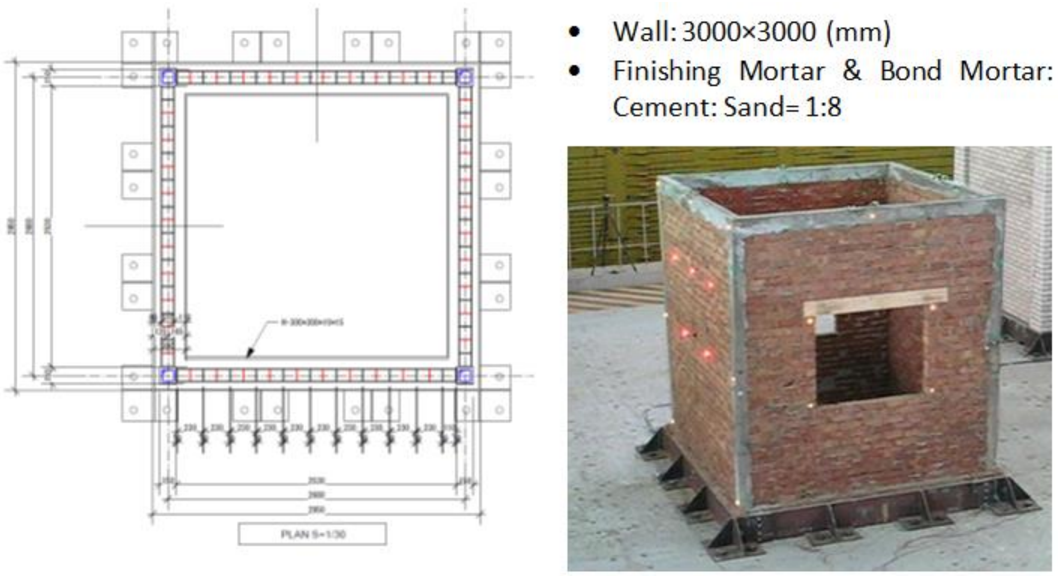


Figure 53 – Outline of Model Structure for Shaking Table Test of Confined Masonry Brick-Walls Construction (National Research Institute for Earth Science and Disaster Prevention (NIED), 2008)

3.5.3.2. Material Properties

The material properties for Pakistan bricks are the same as material properties used in the shaking table test of one brick thick masonry walls construction. Material properties for Japanese bricks are as follows: the compressive strengths were 298kg/cm^2 for bricks, and 25.8kg/cm^2 for mortar test pieces; the elasticity modulus was estimated 83,000 kg/cm^2 for brick, and 25,000kg/cm^2 for mortar in material tests. Frame section for columns and beams are 120mm x 120mm with 4D10 reinforcing bars and stirrups ∅6-150. Weight of house model was about 5ton. The model was built by no skill men, without soaking. Wood lintels were installed on openings (Minowa, et al., 2010).

3.5.3.3. Input Motions

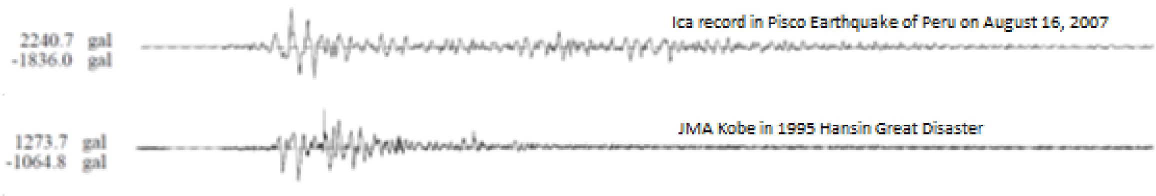


Figure 54 – Input Motion Records for Shaking Table Test of Confined Masonry Brick-Walls Construction (National Research Institute for Earth Science and Disaster Prevention (NIED), 2008)

The input motions for this experiment were Ica record of Pisco Earthquake, August 16, 2007 in Peru and JMA Hansin Great Disaster, Kobe in 1995. Original Ica acceleration record has amplitudes of 0.33g, 62cm/s, 24cm and dominant period of about 3 second, as shown Figure 54. In the shaking table test, Ica record time scale was reduced due to shaking table limitations (Minowa, et al., 2010).

3.5.3.4. Experiment Results

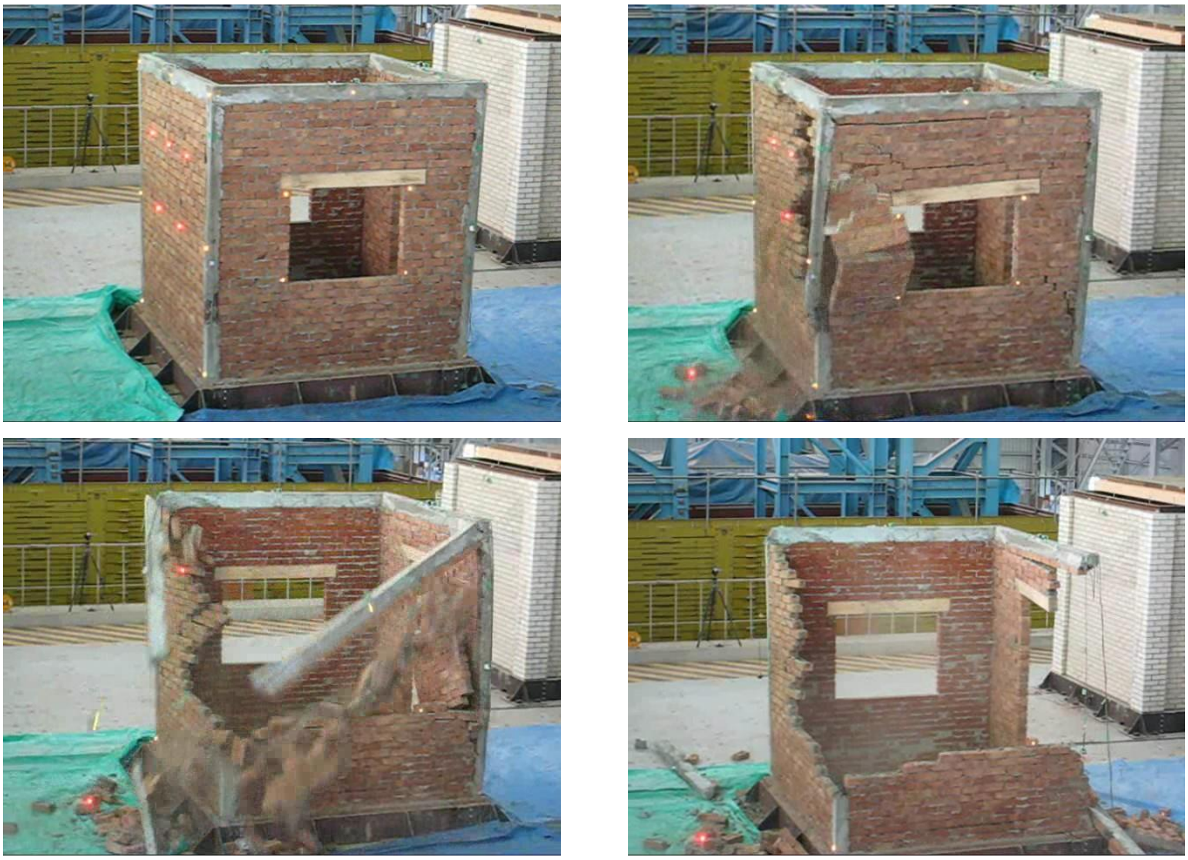


Figure 55 – Sequence Collapsed of Confined Masonry Brick-Walls Construction due to 1995JMA KOBE NS100cm/s (110%) (National Research Institute for Earth Science and Disaster Prevention (NIED), 2008)

When velocity pulse was applied as the first input motion, no damage was observed. The first damage was observed at one frame column when Ica record with time scale 0.1, amplitude 30mm was applied. This input motion produced a maximum velocity 57cm/s with acceleration about 2.2g. After the damages occurred, the house model was shaken by Ica record of time scale 0.6, and velocity 62cm/s, amplitude 140mm. However, damages did not progress largely in the house model. After that, JMA Kobe 110% of velocity 100cm/s was inputted, resulting in the collapsed of the masonry house. However, Japanese bricks remained in tack.

3.5.4. Shaking Table Test of Confined Masonry Brick-Walls Construction with Additional Reinforcement

The shaking table test was conducted at Ponteficia Universidad Catolica Peru (PUCP) using similar specifications of Indonesian materials (Minowa, et al., 2010).

3.5.4.1. <u>Structure Model</u>

PUCP conducted 3 models of houses to test, i.e. (Minowa, et al., 2010):

- Model A had no added reinforcement; it is similar to shaking table test conducted by NIED and Mie University as mentioned before in 3.5.3.
- Model B had a continuous reinforced concrete lintel beam over the door and windows openings, and also steel anchors between walls and columns at three positions. This model has the same concept with the Indonesian earthquake resistant masonry houses as can be seen in Figure 22, page 70 of this dissertation.
- Model C had an external wire mesh covering the surface of the walls, and a mortar cover was placed in East Wall. The wire mesh only wrapped the structure and did not act as ferrocement.

All of this 3 model of houses has dimension approximately 3m each in length and height with wall thickness 105mm. Sand bags were placed over the ring beams to simulate extra roof weight. Therefore the total load of each model was nearly less than 15ton, the limit load for PUCP shaking table. The models are based on Indonesian prevailing practice of non-engineered masonry construction.

Figure 56 – Model Structure for Shaking Table Test of Confined Masonry Brick-Walls Construction with Additional Reinforcement (Minowa, et al., 2010)

3.5.4.2. Input Motions

Three strong motion of earthquake records were used for this experiment: Ica record of Pisco Earthquake, August 16, 2007 in Peru and JMA Hansin Great Disaster, Kobe in January 17, 1995 and LIMA record from Peru earthquake in May 31, 1970 (Minowa, et al., 2010).

3.5.4.3. Experiment Results

The three shaking table test results can be summarized as follows:

Table 4 – Shaking Table Test Results of Masonry Wall Constructions with Additional Reinforcement

	Model A	Model B	Model C
Additional reinforcement	-	continuous reinforced concrete lintel beam and steel anchors between walls & columns	wire mesh covering the surface of the walls
Max. acceleration	2g	2.5g	2.5g
Max. velocity	0.5 m/s	0.5 m/s	0.5 m/s
Results	Heavy cracks; collapsed	Cracks at the bottom of openings; NO separations between walls & columns	No damage

Judging from the above test results, masonry walls which are not using reinforced concrete lintel beams over openings and no anchors between walls and columns has significant damage. The results coincide with past earthquake damages observed (see Chapter 4.1 Learning from Earthquakes Damage). Houses with reinforced concrete lintel beams over openings and anchors have survived although cracks occurred at the corners of the openings. The definition of earthquake resistant building allows cracks or damages, but not endangers human-life.

Shaking table test result of model C showed that wire mesh is a good feature to use as strengthening material of walls. Even if the wire mesh only covered the walls, there is a significant improvement of walls strength, particularly if the ferrocement concept is applied.

3.6. Causes of Masonry Buildings Damage by Earthquakes

As mentioned in Chapter 1, world experience in damaging earthquakes has shown that unreinforced masonry non-engineered constructions are dangerous to human life, often in a relatively small earthquake. In general, the damage and collapse of the non-engineered reinforced masonry buildings during earthquakes are mostly caused by the poor quality of materials and poor workmanship, resulting in, among others poor detailing, poor mortar quality, poor concrete quality, and poor brick-laying. It is a common practice that roof trusses are not strongly anchored to the ring beams (Boen, 2006; Boen, 2007; Boen, 2007; Boen, 2003). In Ref. (Boen, 1978), all those shortcomings were stated; however, with the availability of cement all over Indonesia since approximately 35 years ago and the shifted habit of the uses of lime, replaced by cement, there is a slight improvement in mortar quality (JICA Manado Survey Team, 2009; JICA - Jurusan Teknik Sipil Universitas Negeri Padang, 2009; JICA - Jurusan Teknik Sipil Universitas Negeri Padang, 2010).

The reinforcement of the practical columns and beams are mostly not in accordance with the requirements as mentioned earlier. The reinforcing bars detailing are also not appropriately done for earthquake resistance. The buildings are not designed appropriately and are constructed based on the wrong prevailing practice. Non-engineered masonry buildings failures due to seismic shaking are caused by out-of-plane bending failure of walls, and / or in-plane shear failure and resulted either in total structural collapse or could result in "typical" damages such as walls tear apart; failure at corners of walls; failure at corners of openings; diagonal cracks in walls; walls collapse; failure of connections: weak connection between wall and wall, wall and roof, wall and foundation. These non-engineered masonry buildings failures will be explained in detail in Chapter 4.1 and Chapter 4.3.

Common in any surveyed regions were problems associated with improper use and sizing of structural components. Masonry skills are also missing and for masonry walls to have adequate strength, care must be taken to align the wall both vertically and horizontally, use good mortar and strong bricks or rubble stone. A number of problems have been documented: insufficient or improper foundation; poor mortar (lately improved); poor workmanship; poor quality of bricks; poor concrete mix; additions to existing houses are not adequately tied into the existing house, also creating unsymmetrical configuration. Another major factor contributing to the damage and collapse of such buildings is the lack of maintenance, resulting in deterioration, thus reducing the structural strength (Boen, 2006).

In general, the quality of workmanship for the constructed houses in Indonesia is below average and in many cases poor. This is clearly demonstrated in the reconstruction of Aceh, after the 2004 tsunami (Boen, 2006). Poor quality materials (such as bricks, sand, and timber) combined with poor workmanship (Boen, 2006; Boen & Priyono, 2011) and non-compliance with the Indonesian seismic code resulted in many houses reconstructed so far are below standard.

Unfortunately, all catastrophes in developing countries are mostly due to the collapse of such type of non-engineered constructions. In developing countries, such condition of vulnerability that produces so many disasters is in most cases a result of the poverty that exists in these places. This situation is actually increasing because of uncontrollable population growth, mass urbanization, political instability, debt crisis, some of which vary in intensity from country to country (Boen, 2003).

3.6.1. *Building Materials Quality in Indonesia*

In 1978, the author did state that with the extreme pressures of a great demand for new houses together with a limitation of resources available, including finance, skills and building materials, the tendency has been for the standards to fall from those traditionally established (Boen, 1978).

Past earthquakes damage showed that the damage of the non-engineered constructions were caused by the unavailability of standard building materials and incorrect connection detailing. The lack of maintenance also contributes to the damage and collapse of non-engineered constructions.

Table 5 – Test Results of Building Materials in Indonesia

No		Brick			Masonry Wall	Mortar	Concrete	Plain Bar Reinforcement
		Dimension (mm)		Compressive strength (kg/cm^2)	Compressive strength (kg/cm^2)	Compressive strength (kg/cm^2)	Compressive strength (kg/cm^2)	Tensile stress (kg/cm^2)
1	(NI-10-1978:6)	length	230 / 240		-	-		-
		width	110 / 115					
		height	50 / 52					
2	UNIDO, 1978	length	223.9	27.85	43.8 (1PC:1lime:6sand mortar in 28 days)	151 (1PC:3sand mortar in 28 days)	-	-
		width	100					
		height	56					
3	GRIPS Aceh, 2006	-		60 – 80	-	-	80 - 100	2956
4	JICA North Sulawesi, 2009	-		77.68 (*) 35.76 (**)	-	116.35 (1PC:4sand)	94.324 (1PC:2sand:3gravel) slump 20.3 cm	-
						77.07 (1PC:4sand)	92.399 (1PC:2sand:3gravel) slump 24.3 cm	
						54.4 (1PC:5sand)	102.024 (1PC:2sand:3gravel) slump 22.7 cm	
						99.73 (1PC:5sand)	61.599 (1PC:4sand:1gravel) slump 23 cm	
5	JICA West Sumatra December 2009 - March 2010	length	222.04	23.26 (**) 26.8 (***)	-	-	128.74 (1PC:1sand:2gravel) slump 15 cm	2930
		width	110.5				64.54 (1PC:2sand:2gravel) slump 20 cm	
		height	50.82					
6	JICA North Sumatra & Padang Pariaman, (West Sumatra) October 2011 - March 2012	-		38.9 (*)	-	76.8	56.4	3769
7	Gajah Mada University, 2011	-		25.5 (****)	-	199.8 (1PC:3sand)	-	-
						98.8 (1PC:4sand)		
						48.9 (1PC:6sand)		
8	JICA-Puskim, 2012	-		58 (****)	-	86.4 (1PC:4sand)	259 (1PC:2sand:3gravel: 0.8 water)	3732

For brick compressive strength test, each survey applied different methods; the load was applied in different contact surface area as indicated in Figure 57 below.

(*) the contact surface area is ±100mm x ±100mm

(**) the contact surface area is ±50mm x ±50mm

(***) the contact surface area is ±50mm x ±100mm

(****) the contact surface area is ±100mm x ±100mm (The specimen is half portion of brick which made as 2 layers of brick.)

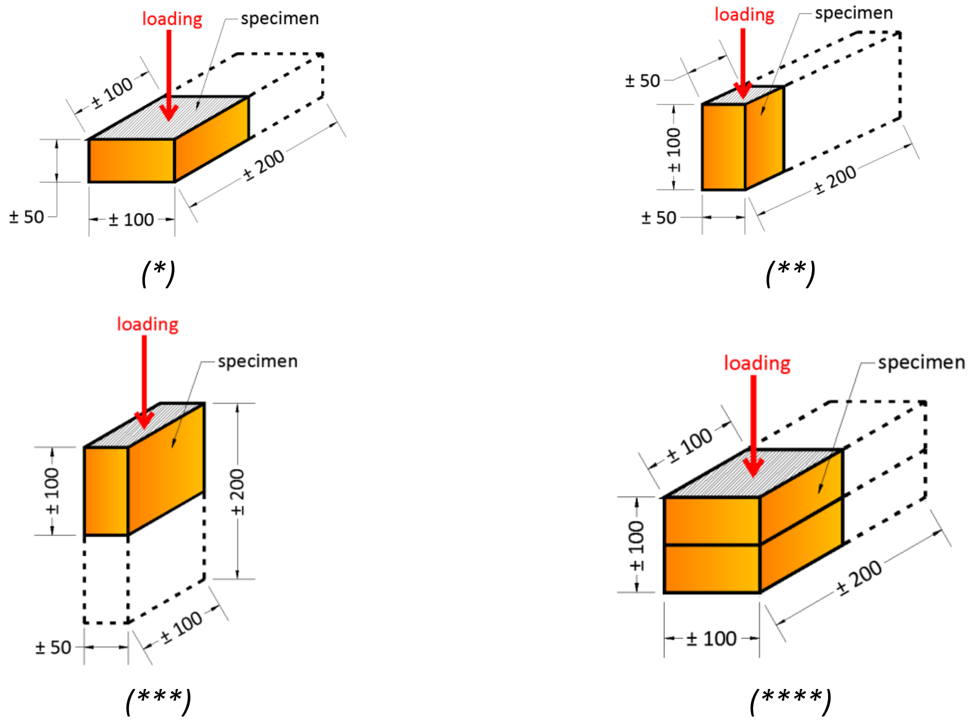


Figure 57 – Various Contact Surface Area for Brick Compressive Strength Test

Surveys and tests in several places in Indonesia were done by JICA and Universities (United Nations Industrial Development Organization (UNIDO), 1978; Building Research Institute, 2006; JICA Manado Survey Team, 2009; JICA - Jurusan Teknik Sipil Universitas Negeri Padang, 2009; JICA - Jurusan Teknik Sipil Universitas Negeri Padang, 2010; JICA - Aneka Asia Buana, PT, 2012; Satyarno, 2011). The objective of these surveys is to know the quality of local building materials. It is important for the people to choose only the qualified building materials, and for the producer to improve the quality of their products.

Summary of surveys and tests of building materials that have been conducted in Indonesia can be seen in Table 5.

Mortar compressive strength is low. It is quite dangerous because masonry wall construction is not a homogeneous material; it consists of brick and the joint. Both have certain strengths and deformations. Normally masonry wall strength is strongly correlated to the strength of bricks, suction rate of bricks, strength of mortar, thickness of mortar, and quality of workmanship. It appears that masonry strength may vary 1/3 power to 2/3 power of mortar strength when the elasticity modulus of brick and mortar are approximately equal (United Nations Industrial Development Organization (UNIDO), 1978). It is better to use mixture 1:4 (Boen, 2005). Concrete compressive strength is very low because the local masons do not measure the water volume of the mixture. Brick dimension is not standard. It is because people's brick factory do not use standard dimensions as regulated by the SNI. Most of them adjust brick sizes based on cost. Apart from that, the shrinkage of the brick during the burning process may cause the brick size to vary. The compressive strengths are not in accordance with the SNI and varied from one survey to others due to the different test method. The size of plain bar reinforcement in the field is also not standard, but the yield and failure stress are still within reasonable limits.

3.6.2. Workmanship Quality – Learning from the Reconstruction of Houses in Aceh post the December 26, 2004 Earthquake

From site observations, it is evident that many of the masons as well as carpenters are "instant" masons and carpenters and lack the necessary skills. This can be observed from the results of their works. Reconstruction of houses in Aceh is evidence that in general, the quality of workmanship is below average and in many cases poor (Boen, 2006).

The quality of the mortar for the masonry walls is also not well controlled and the proportions of the mortar mixes are also not known and are left at the discretion of the foreman and the construction workers. In some places, the quality of mortar sand is good (the mud content is low), but in most cases the mud content is a bit high.

The brick walls are of great concern because very few complied with the rules of good mason workmanship (Building Research Institute, 2006; JICA - Jurusan Teknik Sipil Universitas Negeri Padang, 2010; JICA - Aneka Asia Buana, PT, 2012), which are among others: bricks should be soaked in water prior to construction; the thickness of mortar joints should not be less than 8 mm but not more than 15 mm; bricks should be overlapped on alternate courses and the overlap length for half-brick-thick stretcher bond walls shall not be less than ±0.4 times the length of the brick.

During the reconstruction of 127,400 houses in Aceh (2005-2009) (Boen, 2005; Boen, 2006; Boen, 2006; Ove Arup & Partners Ltd., 2006; Ove Arup & Partners Ltd., 2007; Boen, 2008; Boen, 2008; Boen, 2009; Boen, 2010; Boen & Priyono, 2011), many designs were developed without engineering input. Standard good practice such as the incorporation of ring beams, ties and adequate laps between reinforcement was not shown on construction drawings, and specifications did not adequately cover material quality, testing and workmanship. The facilitators and consultants hired were civil engineers, architects with no specific seismic experience and were unaware of the importance of ductile detailing. The design did not account for seismic loads and the construction drawings prepared were inadequate, not highlighting the importance of ductile details (Boen & Priyono, 2011; Ove Arup & Partners Ltd., 2006; Ove Arup & Partners Ltd., 2007).

Ductility is mentioned and repeatedly stresses on the importance of reinforcing joint details to achieve ductility. Apart from the importance of reinforcing joint detailing, it is equally important to design the concrete components so that during earthquake shaking, the reinforcing bars yield first before the concrete failed. To achieve the ductility, the other factor besides the detailing of reinforcement that must be appropriate, that if a component fail, the reinforcing bars yield first before the concrete fails. The use of deformed bars for main-reinforcement is not appropriate with the used of low concrete quality as mentioned above. An increase in the yield point of the tensile reinforcement decreases the ductility of a section. This can be explained in terms of the greater depth of the compressive block required to balance the tensile force associated with the yield strain in the tension reinforcement (which does not vary significantly with increasing yield stress, f_y), relative to a similar section having tensile reinforcement with the lower f_y. In Indonesia the yield stress of deformed bars is higher than that of plain bars. However, during the reconstruction of houses in Aceh, deformed bars were used because the foreign "consultants" were not aware in this matter. They are used to work in their developed environment, where the minimum concrete quality for structures is more than 300 kg/cm^2, justifying the use of deformed bars. It was unfortunate for the foreign "experts" who spread the news among foreign NGOs in Aceh that it is compulsory to use deformed bars to achieve seismic resistance. This is one of the cases of providing incorrect or conflicting information which caused confusion. From the authors regular surveys in Aceh, only very few NGOs were able to produce good quality concrete using coarse and fine aggregate with the correct mix and therefore were able to produce the necessary concrete strength suitable for deformed bars.

Apart from the deformed bars as explained, the development length of the beam-column joints recommended in manuals and posters regarding the seismic resistant construction of non-engineered construction are 40 cm, namely approximately 40 times the diameter of the reinforcing bars of beams and columns. This also is related to the relatively low quality of concrete compressive strength, namely approximately 100-125 kg/cm^2. This was also misunderstood by "experts" during the reconstruction of Aceh since those experts are familiar with engineered construction based on ACI (American Concrete Institute) only. As is known, in ACI, the minimum compressive stress of concrete is 175 kg/cm^2. Therefore, what is recommended for non-engineered construction is a different context than engineered construction. Having said that, it is important to understand the real problems of non-engineered construction in developing countries before making any remarks in reports and or spreading unfounded news.

3.7. School Buildings Damage by Earthquakes

During the author's surveys and documenting 49 destructive earthquakes as listed in Table 1 many one- or two-story school buildings were also damaged. In fact, school buildings should be made stronger for the following reasons (Boen, 2001; Federal Emergency Management Agency, 1990):

- the collapse of school buildings may cause high toll of human lives,
- most of the occupants are children, who are society's precious resources,
- school buildings may serve as a shelter after the earthquake, thus, they are asset,
- closure of school for a long time may result in community problems, and major school damage may pose long term economic problems,
- the additional cost for new buildings to be earthquake resistant is small, about 1.5 percent of the cost of construction, and
- seismic design, when designed and built properly, for school buildings pays off

As a matter of fact, the design criteria to make earthquake resistant buildings/school buildings are well-known. Such criteria are always written and re-written in many papers during many seminars and conferences concerning earthquake mitigation. However, it is interesting to note that despite of all those knowledge concerning earthquake resistant buildings/school buildings, similar damages/collapses are still occurring every year. What happens to disaster risk reduction in this particular case? What are the major causes?

Apparently, most of the school buildings failures are caused by poor quality of construction, poor workmanship, and improper and inadequate detailing.

Some other factors contributing to the poor quality of construction, poor workmanship, and lack of maintenance in urban as well as rural areas are that: many local governments hire incompetent consultants and incompetent contractors to prevent known problems; in certain cases, government officials lack the desire and/or incentive to act as professionals; many government employees lack administrative support because the supervisors always try to maintain "cordial" relationship with the contractors and their superiors; thus, those officials actually oppose efficient and effective civil servants who try to do an appropriate job and in many instances make life difficult for them if they try to be strict to the contractors; therefore, many civil servants are simply waiting for their retirement and refuse to do anything and do not care to visit construction sites.

There is practically no accountability in those cases due to the so called "government can do no wrong" attitude and thus government employees are immune for their actions or in actions resulting or encouraging professional negligence. To sum up, most of the failures to school buildings during earthquakes are attributed to the lack of consistent professional performance. On top of that, due to lack of accountability, there is also no such thing as cost benefit analysis, giving a false sense of low potential losses. However, in rural areas, apart from the above mentioned, the main cause for poor quality control is that there is a gap between knowledge and application and that despite of our experience from numerous earthquakes, and the growth of our knowledge of aseismic design, the principles are not being communicated to the humble local builders and craftsmen.

3.8. Preventing Further Collapse of Non-Engineered Constructions in Indonesia

Non-engineered masonry buildings with commonly half-brick-thick walls obviously are very rigid and brittle and have no flexibility to absorb earthquake forces. It will collapse suddenly without giving any advanced warning to the dwellers during major earthquakes. On the other hand brick is a cheap construction material, its' ease of production, transportation and construction, has made it very suitable to be used as a construction material. Therefore, the growing trend is to build more and more brick buildings.

In developing countries, masonry construction consists of masonry wall with different kinds of mortar without any reinforcement against earthquake forces. The construction practices in these countries have been developed with the experiences gained by inhabitants over the ages. They have been taking into accounts the weather, availability of materials and the cost but not so for earthquakes. The craftsmen are not trained in trade schools and therefore they are not considered as skilled workers. Hence their method of construction is also poor.

Currently in order to save lives during a large earthquake, the only solution would be to discover some techniques or method how to strengthen these non-engineered constructions. Another solution is to improve rather than to replace the local building materials, at least to the extent so that the building does not collapsed and the dwellers can escape unharmed.

Retrofitting is a corrective measure and therefore, almost all corrective measures can be costly if not done appropriately. It requires considerable expertise and technical know-how when the objective is to achieve better than life-safety performance (Boen, 2008). Retrofitting is needed when the assessment of structural capacity results in insufficient capacity to resist the forces of expected intensity and acceptable limit of damages. For Indonesian non-engineered construction, poor quality of materials and poor workmanship necessitate the retrofitting of the majority of people's houses. Change of the building's function, change of environmental conditions, and change of valid building codes could also be the reasons for retrofitting. For engineered construction, in the design of retrofitting approach, the engineer must comply with the building codes.

The retrofitting method would depend very much on the structural scheme and materials used for the construction of the building in the first instance, the technology that is feasible to adopt quickly and on the amount of funds that can be assigned to the task, which is usually very limited (Arya, et al., 2012).

Retrofitting method for non-engineered construction shall be simple, replicable, and affordable. The method sought and improved materials recommended for strengthening should be low cost and easy to use. The extra cost incurred shall comprise of a small fraction in the total building cost, if otherwise, it will not be accepted by the owners. Its' purpose will definitely be defeated and building with common type of fragile constructions will continue forever. This may be a step to change such buildings into steel, concrete or timber in the course of time with the industrial development and improvement in the people's economy. Till such time it is essential to study the present form of constructions and strengthening them to resist earthquake forces even though they may not be equal in strength as other good materials. This will be a gain and prevent the loss of life and property.

After the July 4, 2000 Bengkulu earthquake, the author did retrofit numerous school buildings and after the September 30, 2009 earthquake retrofitted non-engineered school buildings, religious buildings, and also engineered school buildings and an 8-story hotel building.

Chapter 4 Design Basis of Non-Engineered Constructions

Until few years ago, the design of non-engineered constructions is based on observed behavior of such buildings during past earthquakes and trained engineering judgment. However, with the rapid advancement of the computing power and speed of PCs as well as laptops and the availability of the software, in these last years, it is possible to model non-engineered constructions and perform dynamic analysis.

Guidelines for non-engineered constructions:

- Laboratory tests
- Actual full scale tests → when shaken by earthquakes
- 3D analysis and design

Laboratory test is important, however, learning from earthquake damage, from actual full scale laboratory test results with the real conditions is very important. Actual earthquake damages cannot be duplicated by any laboratory test.

4.1. Learning from Earthquakes Damage

It is said that "Earthquake damage, the mother of earthquake engineering" (Hakuno & Meguro, 1992) and that represents the true expression since it gives a good opportunity to learn from observation of the damages. Observation of structural performance of buildings during an earthquake can clearly identify the strong and weak aspects of the design as well as the desirable qualities of materials and techniques of construction and site selection. Therefore, the study of damage provides an important step in the evolution of strengthening measures for different types of buildings. Every damaging earthquake provides new lessons to be learned.

For non-engineered constructions, from actual site surveys, lessons can be learned from the failures and collapses as well as from buildings that performed well. By studying all the damages and the intact buildings, it can be confirmed that certain methods, procedures or systems are correct. With regard to masonry construction, probably the most universally accepted axiom of earthquake resistant construction is that unreinforced masonry should not be used in earthquake prone countries. Another lesson learnt in that the most important requirements of earthquake resistant construction were to tie all structural elements together so that the building acts as one integral unit when shaken by earthquakes.

The development of man's understanding of the causes and effects of earthquakes has been a gradual process over many centuries. Through careful observation and study of the earth and the damage resulting to his works from earthquakes, man had come to recognize how the great forces of earthquakes act upon his structures. The information which has been obtained bit by bit over the years helps today's engineers to assess the ability of their works and those of their predecessors to withstand this great, destructive force of nature. Without such accumulated information it would not be possible to design modern structures with any reasonable assurance of safety in many parts of the world. All past earthquakes damage reports are another step on the way to a better understanding of earthquakes and their influences.

An earthquake is remarkably effective in pin pointing out structural weaknesses. Most of the structural failures that we observed in past earthquakes were associated with deficiencies in the structure as built, whether caused by design, by lack of supervision, or by improper construction practices (poor materials, poor workmanship).

The investigation of past earthquakes and their effects on various types of structures have contributed significant information to engineers, architects, building officials, and others engaged in extending the knowledge of earthquake engineering. The advancement of theoretical and empirical methods of earthquake resistant design of structures depends upon full-scale tests of structures. Currently the most productive method of obtaining full-scale test information for structures subjected to earthquake motion is field inspection and investigation of structures actually subjected to earthquakes. The field inspection of earthquake damaged buildings is one of the most effective means of obtaining such information. This is particularly true for non-engineered constructions since their earthquake resistant design is mostly based on "observed behavior of such buildings during past earthquakes", and engineering judgment.

In a time span of 40 years, the author has been surveying and documenting 49 damaging earthquakes in Indonesia and in the mass of evidence from past earthquakes, a few facts stand out and although they may be elementary, they are worth reiterating.

The typical damages to non-engineered constructions that confirmed by observing damages of earthquakes during the past 40 years are as follows:

1. Walls tear apart (Figure 58)
2. Failure at corner of walls (Figure 59)
3. Failure at corners of openings (Figure 60)
4. Diagonal cracks in walls (Figure 61)
5. Walls collapse (Figure 62)
6. Failure of connections (Figure 63)
7. Total damage (Figure 64)

There are two major modes of failure of a masonry wall (Mayes & Clough, 1975, p. 123; IAEE, 1980)

- A shear or diagonal tension failure characterized by diagonal cracking
- A flexural or secondary compressive failure characterized by yielding of the tension steel and / or a compressive failure at the compression toe of the wall

Flores – 1982

Halmahera – 1994

Yogyakarta – 2006

Padang – 2007

Figure 58 – Walls Tear Apart

Halmahera – 1994

Bengkulu – 2000

Padang Panjang – 2004

Bengkulu – 2007

Figure 59 – Failure at Corners of Walls

Lombok – 1979

Kerinci – 1995

Flores – 1992

Padang – 2007

Figure 60 – Failure at Corners of Openings

Liwa – 1994

Bengkulu – 2000

Nabire – 2004

Aceh – 2013

Figure 61 – Diagonal Cracks in Walls

Sukabumi – 1982

Padang – 2007

Padang – 2007

Alor – 2004

Figure 62 – Walls Collapse

Biak – 1996

Bengkulu – 2007

Simeulue – 2008

Aceh Tengah – 2013

Figure 63 – Failure of Connections

Halmahera – 1994

Kerinci – 1995

Padang – 2007

Manokwari – 2009

Figure 64 – Total Damage

4.2. Failure Mechanism of Structures

The subsequent step in designing non-engineered constructions is to analyze the typical damages and come up with the structural mechanics explanations of each failure mechanism. In the past, the explanation of failure mechanism obtained by observing behavior of buildings during past earthquakes and engineering judgment. However, since 2001 (Boen, 2001), the author tried to explain the mechanism of failure by computer analysis and the results confirmed the failure mechanism based on study of earthquake damage and engineering judgment.

As outlined in Chapter 4.1 regarding Earthquake Damage and Typical Damage, it is observed that during an intense earthquake, certain effects are seen to occur, roof trusses tend to separate from its supports, the roof covering tends to be dislodged; walls tend to tear apart and if unable to do so they tend to shear off diagonally in the direction of motion; infill walls within steel, reinforced concrete or timber framing tend to fall out bodily unless properly tied to the framing members. From those facts, an analysis of the mechanism of damage is performed (Arya, 1978; IAEE, 1980; Arya, 2007) and is quoted as follows:

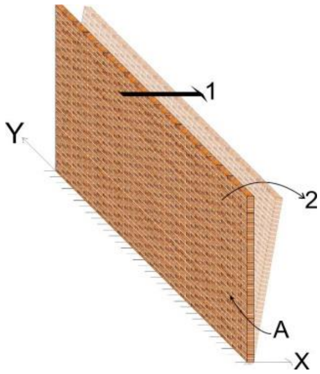
4.2.1. Free Standing Masonry Wall

Consider the free standing masonry walls shown in Figure 65 (a); the ground motion is acting transverse to a free standing wall A. The seismic resistance of the wall is by virtue of its weight and tensile strength of mortar and it is obviously very small. The force acting on the mass of the wall tends to overturn it and the wall will collapse.

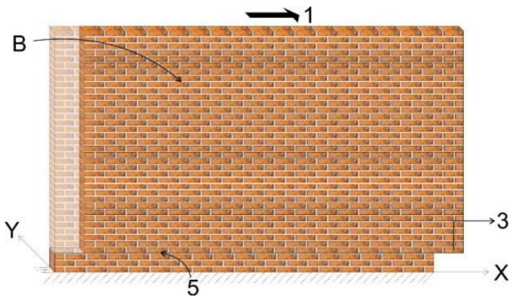
The free standing wall B fixed on the ground in Figure 65(b) is subjected to ground motion in its own plane. In this case, the wall will offer much greater resistance because of its large depth in the plane of bending. Such a wall is termed a <u>shear wall.</u> The damage modes of an unreinforced shear wall depend – on the length-to-width ratio of the wall.

A wall with small length-to-width ratio will generally develop a horizontal crack due to bending tension and then slide due to shearing. A wall with moderate length-to-width ratio and bounding frame diagonally cracks due to shearing as shown at Figure 65 (c).

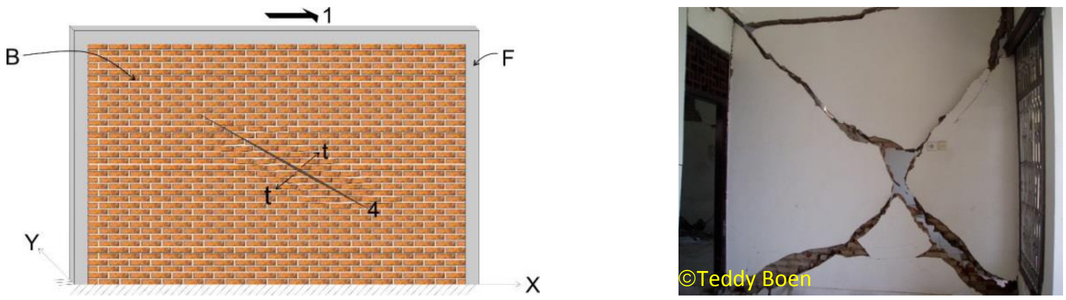
A wall with large length-to-width ratio, on the other hands, may develop diagonal tension cracks at both sides and horizontal cracks at the middle as shown at Figure 65 (d).



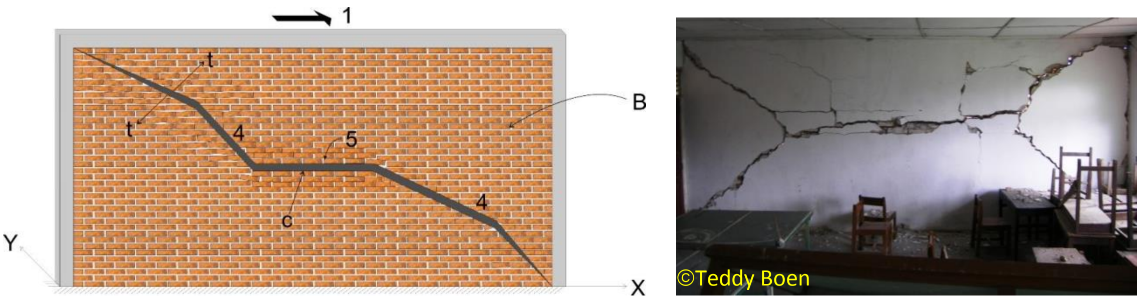
(a) Perpendicular force causing overturning



(b) Unreinforced wall B with small length-to-width ratio



(c) Wall B with moderate length-to-width ratio



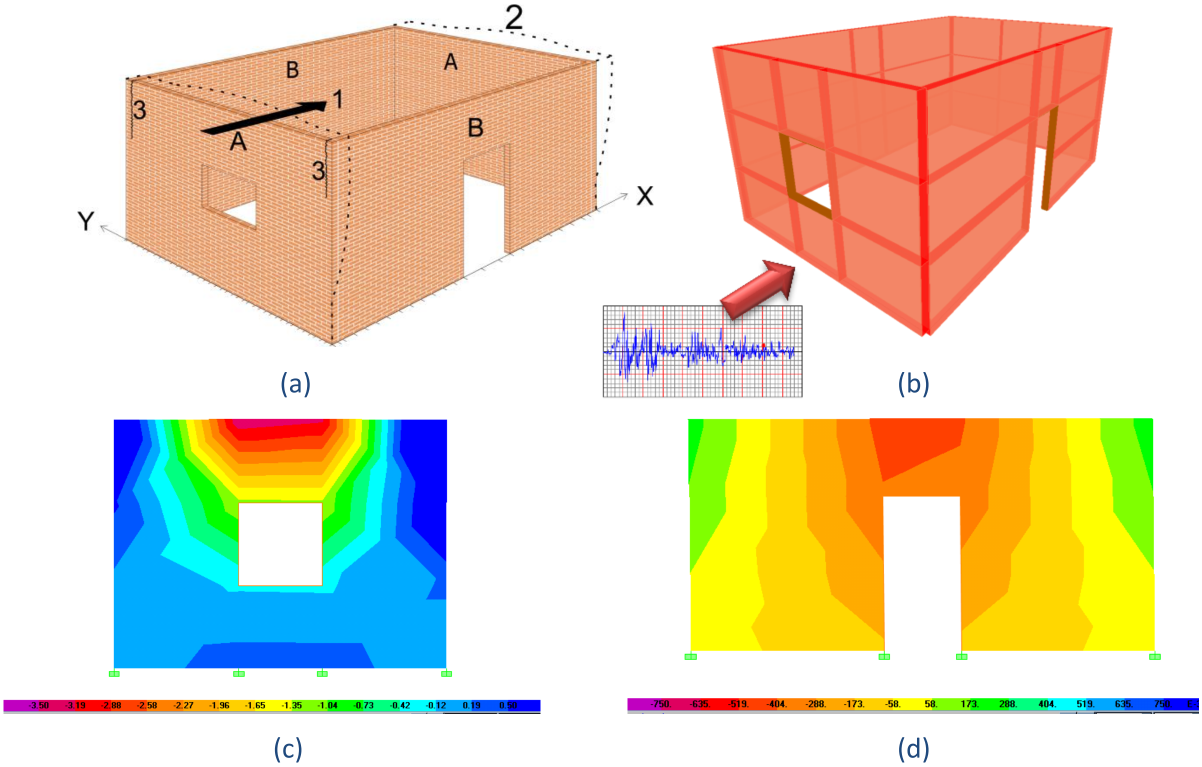
(d) Wall B with larger length-to-width ratio

A = Wall A; B = Wall B; F = Framed; 1 = Earthquake force; 2 = Overturning; 3 = Sliding; 4 = Diagonal cracking; 5=Horizontal cracking

Figure 65 – Failure Mechanism of Free Standing Walls

4.2.2. Wall Enclosure without Roof

Now consider the combination of walls A and B as an enclosure shown in Figure 66. For the X direction of force as shown, walls B act as shear walls and, besides talking their own inertia, they offer resistance against the collapse of walls A as well. As a result walls A now act as vertical slabs supported on two vertical sides and the bottom plinth. Walls A are subjected to the inertia force of their own mass. Near the vertical edges, the wall will carry reversible bending moments in the horizontal plane for which the masonry has little strength. Consequently cracking and separation of the walls may occur along these edges shown in the Figure 66.



1 = Earthquake force; 2 = Bending of Wall A; 3 = Bending cracks at ends of wall A

Figure 66 – (a) Failure Mechanism of Wall Enclosure without Roof; (b) 3D-model; (c) Tensile Stresses Pattern due to Out-of-plane Loading; (d) Shear Stresses Pattern due to In-plane Loading

It can be seen that in the action of walls B as shear walls, walls A will act as flanges connected to walls B acting as web. Thus if the connection between walls A and B is not lost due to their bonding action as plates, the building will tend to act as a box and its resistance to horizontal loads will be much larger than that of walls B acting separately. Most unreinforced masonry enclosures, however, have very weak vertical joints between walls meeting at right angles due to the construction procedure involving toothed joint which is generally not properly filled with mortar. Consequently the corners fall and lead to collapse of the walls. It may also be easily imagined that the longer the walls in plan, the smaller will be the support to them from the cross walls and the lesser will be the box effect.

The mechanism of damage now can be confirmed by modeling using commercial softwares.

4.2.3. Two Walls with Roof

In Figure 67 (a), roof slab is shown to be resting on two parallel walls B and the earthquake force is acting in the plane of the walls. Assuming that there is enough adhesion between the slab and the walls, the slab will transfer its inertia force at the top of walls B, causing shearing and overturning action in them.

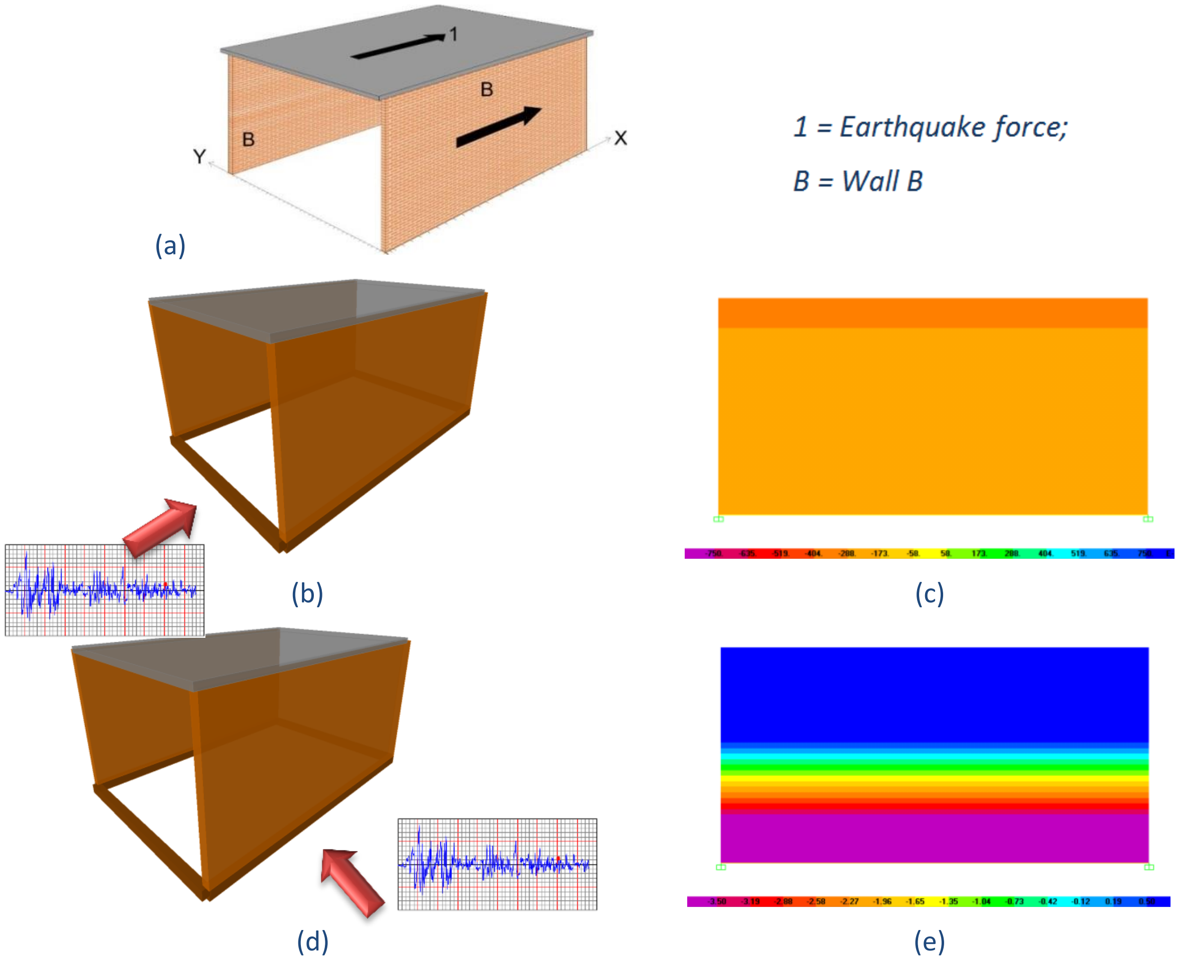
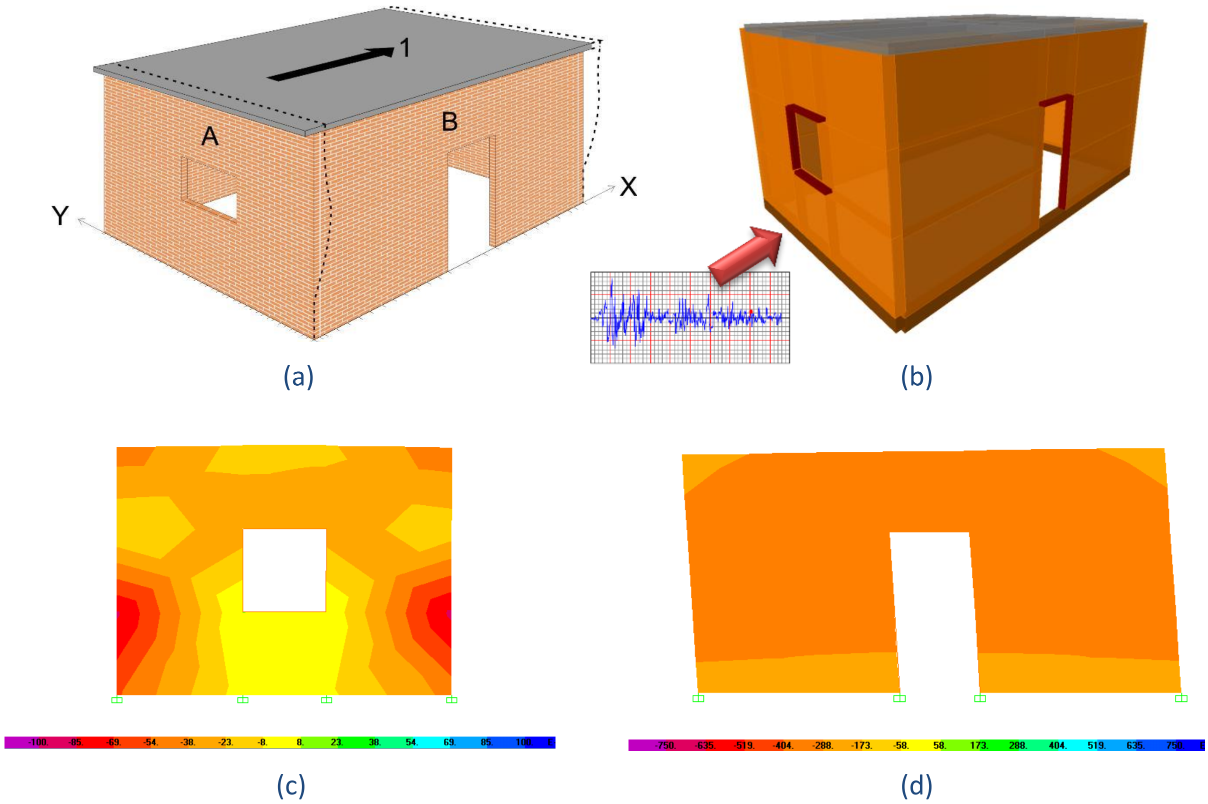


Figure 67 – (a) Failure Mechanism of Two Walls with Roof; (b & d) 3D-model & Load Direction; (c) Shear Stresses Pattern due to In-plane Loading; (e) Tensile Stresses Pattern due to Out-of-plane Loading

To be able to transfer its inertia force to the two end walls, the slab must have enough strength in bending in the horizontal plane. This action of slab is known as diaphragm action. Reinforced concrete or reinforced brick slabs have such strength inherently and act as rigid diaphragms. However, other types of roofs or floors such as timber or reinforced concrete joists with brick tile covering will be very flexible. The joists will have to be connected together and fixed to the walls suitably so that they are able to transfer their inertia force to the walls. At the same time, walls B must have enough strength as shear walls to withstand the force from the roof and its own inertia force. Obviously, the structure shown in Figure 67, when subjected to ground motion perpendicular to its plane, will collapse very easily because walls B have little bending resistance in the plane perpendicular to it. In long barrack type buildings without intermediate walls, the end walls will be too far to offer much support to the long walls and the situation will be similar to the one just mentioned above.

4.2.4. Wall Enclosure with Roof



1 = Earthquake force; A = Wall; B = Wall B

Figure 68 – (a) Failure Mechanism of Wall Enclosure with Roof; (b) 3D-model; (c) Tensile Stresses Pattern due to Out-of-plane Loading; (d) Shear Stresses Pattern due to In-plane Loading

Now consider a complete wall enclosure with a roof on the top subjected to earthquake force acting along X-axis as shown in Figure 68. If the roof is rigid and acts as a horizontal diaphragm, its inertia will be distributed to the four walls in proportion to their stiffness. The inertia of roof will almost entirely go to walls B since the stiffness of the walls B is much greater than the walls A in X direction. In this case, the plate action of walls A will be restrained by the roof at the top and horizontal bending of walls A will be reduced. On the other hand, if the roof is flexible, the roof inertia will go to the wall on which it is supported and the support provided the plate action of walls A will also be little or zero. Again the enclosure will act as a box for resisting the lateral loads; this action is decreasing in value as the plan dimensions of the enclosures increase.

4.2.5. Roofs and Floors

The earthquake-induced inertia force can be distributed to the vertical structural elements in proportion to their stiffness, provided the roofs and floors are rigid to act as horizontal diaphragms. Otherwise, the roof and floor inertia will only go to the vertical elements on which they are supported. Therefore, the stiffness and integrity of roofs and floors are important for earthquake resistance. The roofs and floors, which are rigid and flat and are bonded or tied to the masonry, have a positive effect on the wall, such as the slab or slab and beam construction directly cast over the walls or jack arch floors or roofs provided with horizontal ties and laid over the masonry walls through good quality mortar. Others which simply rest on the masonry walls will offer resistance to relative motion only through friction, which may or may not be adequate depending on the earthquake intensity. In the case of a floor consisting of timber joists placed at a center to center spacing of 20 to 25 cm with brick tiles placed directly over the joists and covered with clayey earth, the brick tiles have no binding effect on the joists. Therefore, relative displacement of the joists is quite likely to occur during an earthquake, which could easily bring down the tiles, damaging property, and causing injury to people. Similar behavior may be visualized with the floor consisting of precuts reinforced concrete elements not adequately tied together. In this case, relative displacement of the supporting walls could bring down the slabs.

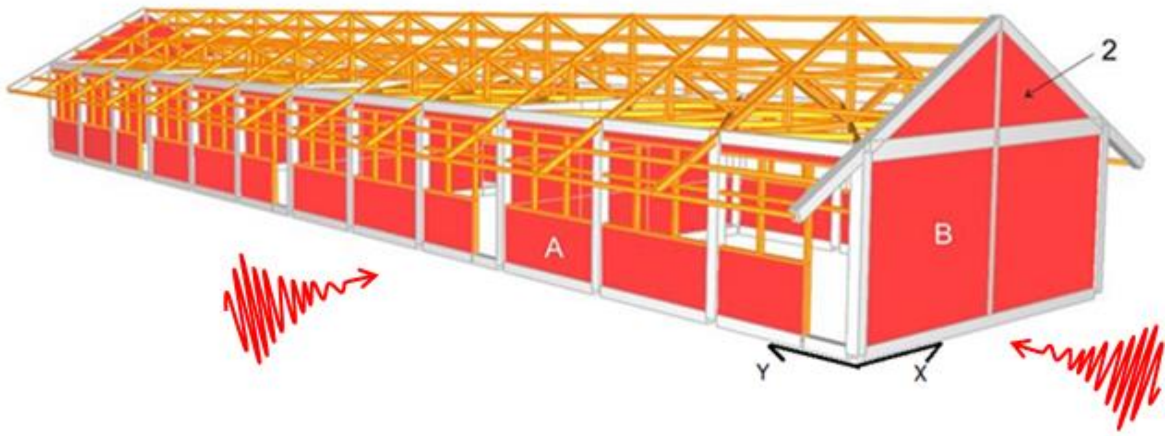
4.2.6. Long Building with Roof Trusses

Consider a long building with a single span and roof trusses as shown in Figure 69. The trusses rest on walls A. Walls B are gabled to receive the purlins of the end bays. Assuming that the ground motion is along the X-axis, the inertia forces will be transmitted from sheeting to purlins to trusses and from trusses to walls A.

The end purlins will transmit some forces directly to gable ends. Under the seismic force the trusses may slide on the walls unless anchored into them by bolts. Also, walls A, which do not get much support from walls B in this case, may overturn unless made strong enough in the vertical bending as a cantilever or other suitable arrangement, such as adding horizontal bracing between the trusses, is made to transmit the force horizontally to end walls B.

When the ground motion is along Y direction, walls A will be in a position to act as shear walls and all forces may be transmitted to them. In the case, the purlin act as ties and struts and transfer the inertia force of the roof to the gable ends.

As a result the gable ends may fail. When the gable triangles are very weak in stability, they may fail even in small earthquakes. Also, if there is insufficient bracing in the roof trusses, they may overturn even when the walls are intact.



1 = Earthquake force; 2 = Gable end; A = Wall A; B = Wall B

Figure 69 – Long Building with Roof Trusses

4.2.7. Shear Wall with Openings

Shear walls are the main lateral earthquake resistant elements in many buildings. For understanding their action, let us consider a shear wall with three openings shown in Figure 70. Obviously, the piers between the openings are more flexible than the portion of wall (sill masonry) or above (spandrel masonry) the openings. The deflected form under horizontal seismic force is also sketched in the Figure 70.

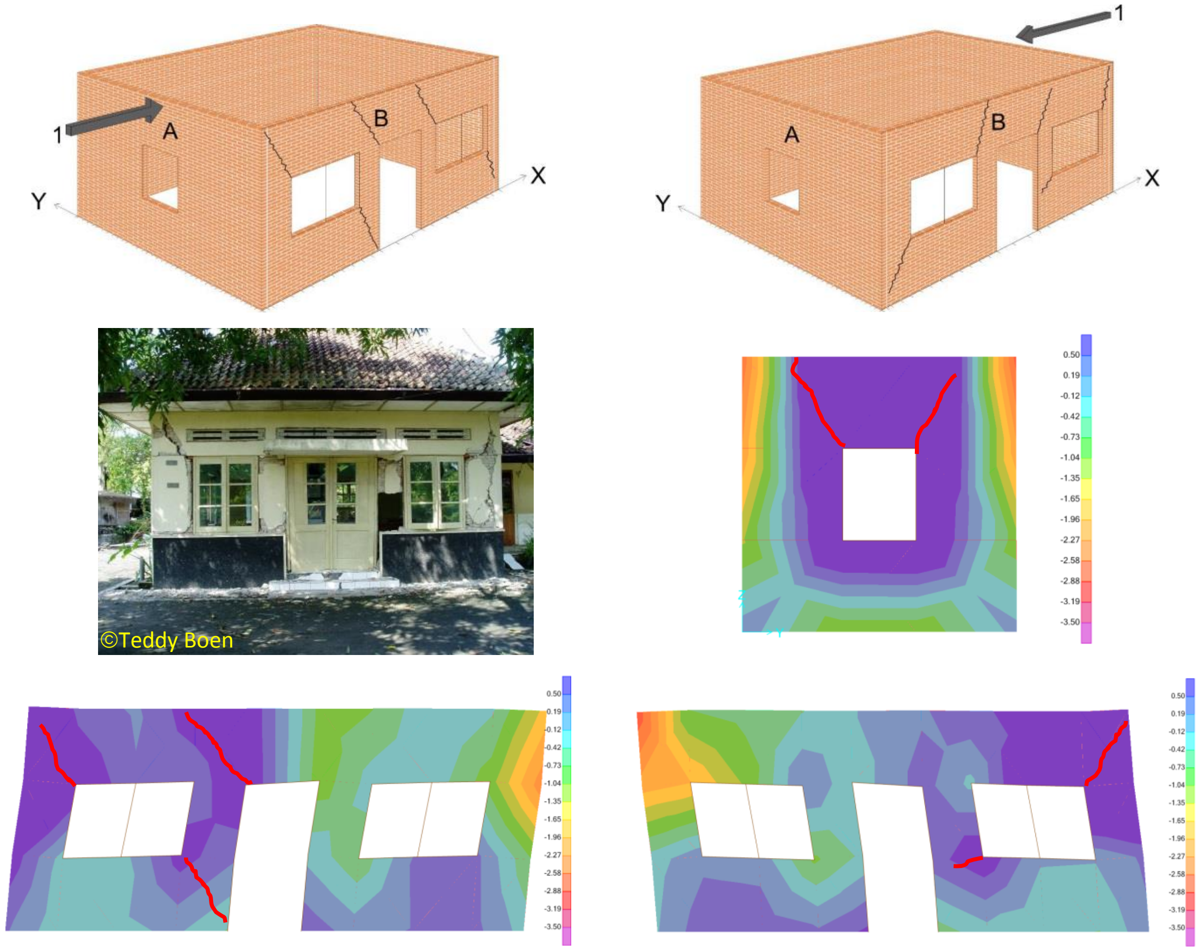


Figure 70 – Cracks and Tensile Stresses Pattern of a Shear Wall with Openings – Red Lines: Probability of Cracks

The sections at the level of the top and bottom of openings are found to be the worst stressed in tension as well as in compression and those near the mid-height of piers carry the maximum shear. Under reversed direction of horizontal loading the sections carrying tensile and compressive stresses are also reversed. Thus it is seen that tension occurs in the jambs of openings and at the corners of the walls.

4.3. Loading Causing Damage of Masonry Buildings

In every earthquake damage of masonry buildings, cracks always occur. Cracks develop in areas of high stress concentrations, e.g. corners of openings (door and window frames), corners of walls, intersections of perpendicular walls, base of walls. As damage to a masonry building progresses, crack size increases during reverse cycles of ground motion. When the cracks fully developed, each wall becomes an assemblage of irregularly shaped wall segments or broken masonry blocks. Unreinforced half-brick-thick masonry walls tend to become unstable upon initiation of cracks through the section. However, thick masonry walled buildings do not loose its stability when first cracks in the walls develop.

In general, failure mechanism of non-engineered masonry buildings due to seismic shaking is mainly caused by out-of-plane bending failure of walls, and / or in-plane shear failure. Out of plane wall displacements pose the largest collapse threat to unreinforced masonry (URM). Non-load bearing walls often are the first to collapse because there is often little or no restraint provided by the roof or floor connections and no tributary loads. Gabled walls collapse simply due to their additional greater height to thickness ratio.

4.3.1. Out-of-Plane Plane Bending Failure of Walls

Primary factors affecting out of plane stability (overturning) of badly cracked walls:

- The absolute thickness and slenderness (height to thickness ratio) of the wall.
- Other restraints that may limit the deflection at the top (connection at the floor or roof line) or sides (perpendicular walls) of each block formed in the wall. Vertical cracks may develop such that perpendicular walls provide little stabilizing effects.
- Added gravity loads from roof or floor framing.

Primary factors affecting collapse of a bearing wall are its absolute thickness, its slenderness ratio, and the degree of restraint at the top. Out-of-plane failure in buildings with thin walls and high height to thickness ratios, where the initiation of cracks through the wall section; does in fact threaten the stability of the wall. The provision of restraints at the tops of walls can provide significant additional out-of-plane stability and adds redundancy to the structural systems. The longitudinal dimension of a wall, or independent cracked block, may have little effect on its potential for overturning or collapse, unless the top of the wall is not anchored to cross walls at the floor or roof level. Connections that tie the tops of walls to straight-sheathed roof or ceiling systems, to bond beams, or simply parallel wall can tremendously improve the out-of-plane rocking stability of a wall. Adequate connection between the walls and either the roof beams and the lack of a positive connection can allow a load bearing wall to progressively move out from under the beam.

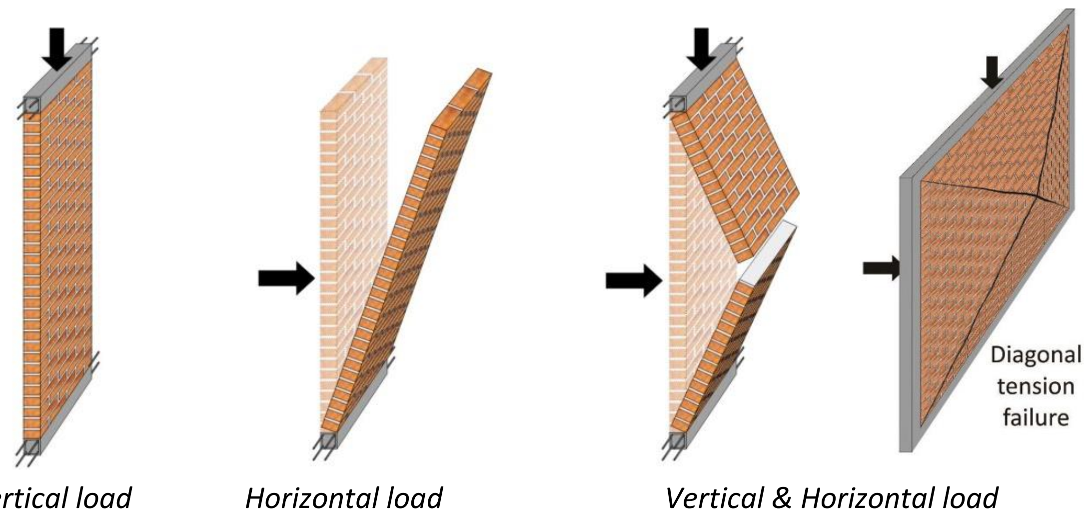


Figure 71 – Behavior of URM Wall Subjected to Vertical and Out-of-Plane Lateral Load

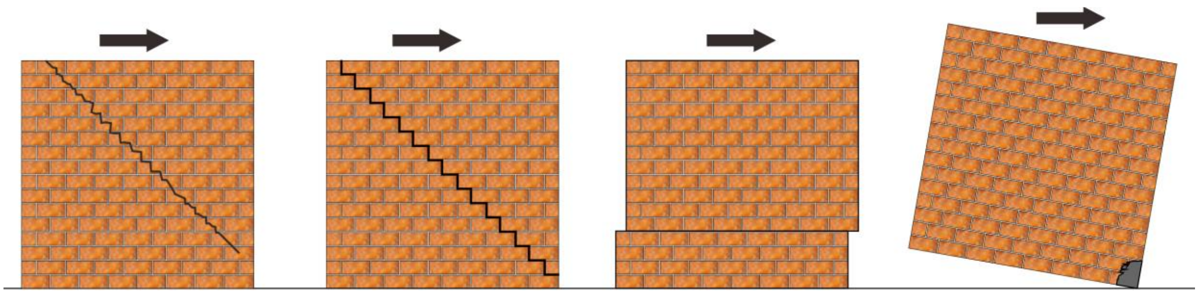
4.3.2. In-Plane Shear Failure of Walls

In-plane shear damage to walls will increase substantially during large seismic movements. Diagonal cracks may develop in sections of the walls with no opening. The movement of wall blocks may be exacerbated by gravity and friction as shaking continues. Broken segments near the end of walls are susceptible to non-reversing sliding along diagonal cracks. The in-plane movement of blocks can be particularly problematic from the point of view of repair.

The main in-plane failure mechanism of URM walls subjected to earthquake actions are summarized as following (Elgwady, et al., 2002; City University London & Pell Frischmann Group, 2013; Ghiassi, et al., 2012):

- Walls loaded with vertical and horizontal loads can fail in shear if the tensile resistance of the masonry wall material is exceeded. Prior to reaching its tensile stress, diagonal cracks are developed in the wall. In case of strong bricks weak mortars, the cracks are stair steps, passing through the mortar. In case of a weak bricks and strong mortar, the cracks pass through the bricks. Shear failure can occur in walls with aspect ratio 1:1 up to 1:2 (Figure 72(a)). This type of failure is considered as brittle behavior with sudden loss of strength.
- Sliding shear: this failure mechanism can occur if the wall has poor shear strength when shaken by earthquakes. Poor shear strength can be caused by poor quality of mortar and horizontal cracks along the bed joints will develop This can occur for walls with aspect ratio of 1 : 1 up to 1 :1.5 (Figure 72(b)).
- Bending failure can occur in walls where the shear resistance is improved (high moment to shear ratio). In walls with an aspect ratio of more than 2:1 and a small vertical load (low compressive forces), a bending failure will occur and not a shear failure. Usually, bending failure resulted in large deformation and the wall becomes unstable. When rocking continues in several cycles, toe crushing can occur (Figure 72(c)). If the vertical load is small, the wall can rock like a rigid body.
- Sliding shear and bending type of failure are considered as ductile behavior if there is no considerable loss of strength.

The in-plane wall motions do not typically have the catastrophic consequences as the out-of-plane motions. In-plane capacity of a masonry wall is considered higher that the out-of-plane capacity. Therefore, the diaphragm is used to redirect seismic forces from out-of-plane in one wall to in-plane on perpendicular wall, improving the seismic performance.



(a) Shear Failure if Weak Bricks – Strong Mortar | *(b) Shear Failure if Strong Bricks – Weak Mortar* | *(c) Sliding Failure* | *(d) Rocking Failure*

Figure 72 – In-Plane Failure Mechanism of Laterally Loaded URM Wall

The above explanations in general applies to “thin” masonry walls, For half-brick-thick masonry walls, the walls usually becomes unstable as soon as cracks develop. For thick masonry walls, the post elastic behavior of masonry buildings explicitly addresses the stability of a structure. Masonry cracks at relatively low level of seismic shaking due to the low strength of the materials, but the cracks typically form along mortar joints perpendicular to the plane of the wall. This makes unlikely the slumping of material under gravity loads.

It is often assumed that an unreinforced masonry structure is only safe while it is largely undamaged, that is, without substantial cracking. Once cracks have developed, the usual analysis proceeds to note that the material has lost its continuity and strength and, therefore, the building is unsafe because the damage means the building is at a point of imminent collapse. This is true for half-brick-thick masonry walls in Indonesia as observed from past earthquake damage. From past earthquake surveys, it was observed that a thick walled masonry building is NOT unstable after cracks have fully developed. A thick walled masonry building still retains considerable stability characteristics even in a fully cracked state.

Usually most of the walls of historic masonry buildings are thick and possess some ductility and can sustain the forces without total collapse.

4.4. Modeling Masonry Buildings – Wall Bearing Construction

The computing power and speed of desktop / laptop computers has increased at a breathtaking rate over the last 15 years. The availability of softwares make it practical for engineers to perform static and dynamic analysis of structures quickly and efficiently (Boen, 2001; Boen, 2003; Boen, 2007).

Masonry wall bearing structures are typically shear wall structures. Therefore, the basic principles, hypothesis and mathematical modeling used for modeling reinforced concrete shear wall structures are also applicable for masonry shear wall structures, provided the models for reinforced concrete shall be modified taking into account the specific mechanical characteristics of masonry and the materials.

Masonry wall buildings are generally box type constructions with brick walls as the load bearing walls or partition walls. The wall panels are modeled as shell elements. Shell elements behavior is a combination of a plate bending behavior plus a membrane behavior.

The most crucial point in modeling of walls is to include the walls in the structural model. Walls are best modeled using membrane or shell finite element. Modeling wall or shear core as equivalent columns is NOT a good idea. For elastic finite element model, the finer the element mesh, the more accurate the analysis. However, if the analysis is run with a cracking material and if the finite element meshes are progressively refined, the cracking will be confined to a few smaller and smaller elements, so the maximum calculated strain progressively increases. This does not happen in an actual structure, so refining the finite element mesh gives wrong results. There are ways to account for this, using fracture mechanics principles, but they are too complex for most practical applications (Powell, 2013).

Confined masonry buildings are occasionally modeled as frame-type structures composed only of the frame elements, completely neglecting the influence of the masonry (Bozorgnia & Bertero, 2004). This “frame-only” approach, though often believed to be conservative, is in fact un-conservative and erroneous, for the following reasons:

- It greatly increases the building’s calculated period of vibration, thereby decreasing (usually) its calculated seismic inertia forces.
- It gives incorrect estimate of the internal distribution of shears among wall elements, and of the building’s plan center of rigidity.
- It gives significant errors in calculating the lateral resistance of the building.

Buildings with essentially un-perforated shear walls can usually be modeled adequately considering the walls as solid panels. This approach, however, is inadequate for the design and analysis of masonry buildings having structural walls with large openings. For such buildings, two alternatives are available:

- The walls can be modeled with solid panels having equivalent axial, shear, and flexural stiffness. This equivalent stiffness should be determined by separate finite element modeling of the original panel.
- The building can be modeled using finite elements with appropriately placed openings.

There are two basic systems in modeling masonry walls:

1. Bearing wall
    a. The walls support vertical as well as lateral load.
    b. In the Indonesian type of confined masonry bearing wall constructions, the masonry walls, and the practical columns and beams are designed to support vertical load in compression as well as lateral earthquake loads. The reinforced concrete confinement and the walls develop substantial shear forces when shaken by earthquakes and also subject to bending.
2. In-filled frame
    a. Always include a vertical as well as lateral load carrying frame of concrete or steel beams and columns. Wall panels are placed within the frame and is called filler wall and the assembly is called in-filled frame. To be effective at resisting in-plane lateral loads, the infill must be in contact with the surrounding frame.
    b. The vertical load is taken up by the frame and when subject to lateral loads (earthquake or strong wind), the frame and the filler walls absorb the shear.

The main function of non-structural walls such as parapets and interior partitions is to provide enclosure in buildings. Therefore, the contribution of these walls to the overall structural resistance should be minimal. However, if the partitions are connected to the load carrying frame, the partitions must be modeled and analyzed similar to bearing walls.

Figure 73 – Example of Bearing Wall (left) and In-filled Frame (right)

4.5. Analysis and Design using Commercial Software

In many European countries and the US, elastic method has been used since a long time for the verification and design of masonry structures. As is known, when using elastic method, the seismic forces are distributed based on elastic theory and it will be practically impossible that all structural elements reached the permissible stresses simultaneously. Therefore, there will be only few sections of masonry walls of the entire building which will be fully stressed when subjected to the design loads. This is good enough until a more refined limit state verification method is established for masonry wall bearing structures that is easy to apply.

The purpose of the analysis is not to simulate the actual behavior, but to get reliable information that there is a correlation between the observed damages and the results of the analysis. The correlation is not perfect, but is good enough to get a good idea to build appropriate non-engineered constructions that can withstand earthquakes.

For the purpose, non-engineered masonry buildings are verified using 3-D models utilizing commercial software (Boen, 2001; Boen, 2003; Boen, 2007), like SAP2000. The use of commercial software for analyzing non-engineered constructions achieves recognition from software developers; Computer & Structures, Inc. (CSI) in 2004 as can be seen in Figure 74.


TECHNOLOGY FOR A SAFER WORLD PUT INTO ACTION

By Mary Harkin, CSI Public Relations Coordinator

CSI Software used for Earthquake Resistant Housing in developing countries.

How technology can impact the lives of ordinary people in rural areas of underdeveloped countries can be extraordinary. Application of software donated by CSI is having is having that kind of an impact ~ enhancing seismic safety in developing countries such as Indonesia.

A house is meant for safe human habitation, a structure with a special purpose for people to have shelter from the elements and a sense of protection and comfort. But living in a dwelling that could be potentially dangerous due to its location in an earthquake zone is an unsettling proposition.

Every year earthquake occur in various areas of Indonesia. Teddy Boen, a prominent Indonesian Structural Engineer and the Director of the World Seismic Safety Initiative, has spent over 30 years working on the subject of low-income housing projects termed as "non-engineered" construction in developing countries. Mr. Boen has tried to introduce "the engineering of non-engineered buildings" and has carefully studied the performance of such structures in past earthquakes to help develop guidelines for the earthquake resistant design of under privileged people's housing.

Mr. Boen has used CSI software, SAP 2000 and ETABS in his research and analysis to correlate his field observations with the results from the simulations produced by the 3D mathematical models of the software. Results of his work were published in his paper entitled "Earthquake Resistant Design of Non-Engineered Buildings in Indonesia".

a. Actual Column Damage

b. Analysis result of 3D Model

Approximately 70 percent of buildings inventory in Indonesia consists of urban and rural non-engineered buildings. These buildings are built according to tradition, their types suiting the culture and materials available in that area. Many of these structures are masonry buildings and were built with poor workmanship and poor quality of materials. Unfortunately, all catastrophes in developing countries such as Indonesia are mostly due to the collapse of such type of non-engineered buildings during an earthquake.

The study of earthquake damage as well as field inspection provides new lessons to be learned for engineers in designing structures that will withstand earthquakes. The availability of software such as SAP2000 makes it practical for engineers to perform static and dynamic analysis of structures quickly and efficiently. These buildings can be re-engineered by verification using the powerful technology available in SAP2000 and ETABS.

By observing the damage of structures in a real earthquake and then being able to simulate such damage using modern numerical methods along with powerful computer technology and graphics allows engineers to identify weaknesses in the structural behavior using a mathematical model during the design phase. These weaknesses can then be addressed before the structure is built. Now that is a story about technology for a safer world!

Figure 74 – CSI Software used for Earthquake Resistant Housing in Developing Countries (CSI Berkeley, 2004)

The method of analysis is still based on Strength Based Design (SBD) which assumed all elements are linear-elastic. A linear elastic analysis model is much simpler than a non-linear inelastic one. It is computationally simple and allows superposition for load combinations. The analysis for each load combinations can be done separately; the results can be multiplied by load factors, and combine with any other load.

Recently non-linear analysis of non-engineered structures was performed (Boen, 2007). However, for masonry constructions, there is usually no need for sophisticated non-linear dynamic analysis to be carried out for seismic resistance verification (Tomazevic, 1999) since it is difficult to determine the force-deformation (F-D) relationship for masonry, and also it is difficult and complicated for common people to perform non-linear dynamic analysis. The fact is that more sophisticated analysis is not the solution. The goal is to control the behavior with a high level of confidence, not absolute certainty.

Strength Based Design refers to the process of focusing on:

- The strength of the material
- The individual connection behavior
- The overall structural configuration, e.g. addition of shear walls or diaphragms

It provides sufficient strength for the structural elements to resist the forces generated by the elastic response of the building during a design level earthquake. The forces generated during seismic events will exceed those generated during the design level. However, it is assumed that the non-linear deformations of the material and connections have sufficient ductility to dissipate the additional energy from larger earthquakes. Strength Based Design usually only assesses the possible consequences of extreme deformations by assessing the elastic deformations under larger than design earthquakes.

In strong earthquakes, most structures are expected to yield. Strength Based Design allows little or no yield, and is rarely used for strong earthquakes. SBD provides safety by ensuring that the structure strength exceeds all considered loads (not necessarily all possible loads). When SBD is used for earthquake load, the most likely method is response spectrum analysis.

Thin walled unreinforced masonry construction can fail catastrophically simply due to gravity conditions shortly after the material is cracked through the section and blocks have formed. Thick walled masonry construction is observed to be capable of sustaining deflections well beyond the elastic limit of the material.

The “structural ductility” of a building system (NOT material ductility); meaning the capacity of a building to deform beyond the deflections at the elastic limit of the material while the building still maintains its load carrying capacity; is a critically important characteristic of the seismic design of a building. Thick walled masonry buildings can exhibit substantial structural ductility even though the brick itself is brittle. The structural ductility of a masonry building is proportional to the thickness of the walls.

As an example, the results of the analysis of a non-engineered house and a school building in Indonesia are shown below. From the results of analysis it can be seen that there is a good correlation between actual earthquake damage and results of analysis.

4.5.1. Dwelling House with Confinement

Figure 75 – 3-D Model Non-Engineered House with Practical Column

The analysis is elastic using response spectrum based on the Indonesian Seismic Code 2012 and ASCE7-10 Table 12.2-1 for Bengkulu area (Hard soil; PGA = 0.42g; Importance Factor = 1.0, R = 1.5).

Below are the results of 3D-analysis using SAP2000 v15. The blue color (▬) indicates the stresses exceed the permissible stress.

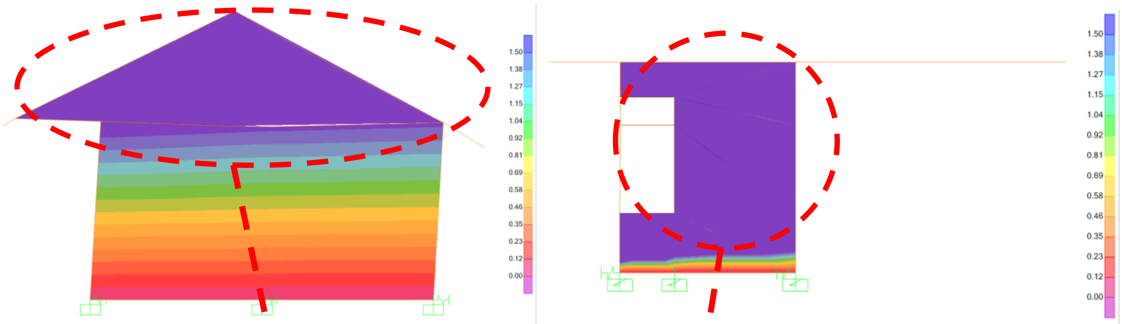


Figure 76 – Tensile Stresses Pattern at Masonry Wall

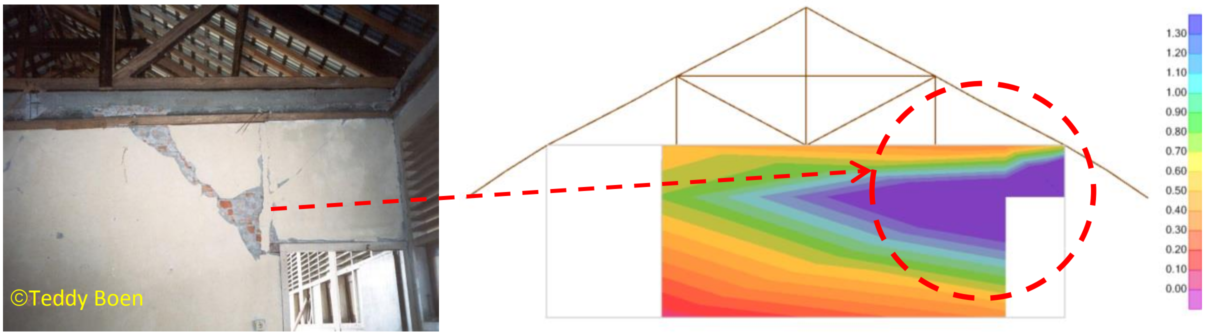
Figure 77 – Actual Damage – Bengkulu Earthquake (2000)

4.5.2. One-Story Masonry School Building with Confinement

Figure 78 – Actual School Building and 3D Model for Analysis

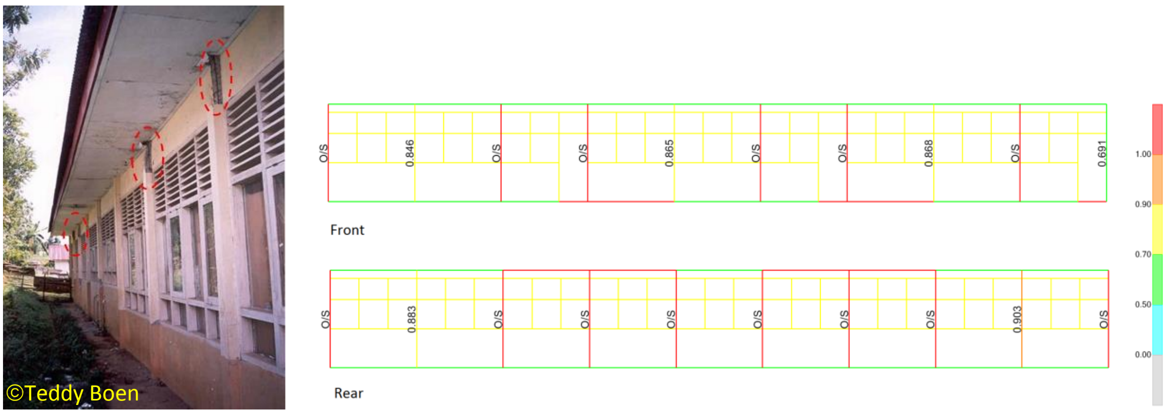
The analysis is elastic using response spectrum based on the Indonesian Seismic Code 2012 and ASCE7-10 Table 12.2-1 for Bengkulu area (Hard soil; PGA = 0.42g; Importance Factor = 1.5, R = 1.5).

Below are the results of 3D-analysis using SAP2000 v15. The blue color (■) indicates the stresses exceed the permissible stress.



(a) (b)

Figure 79 – (a) Actual Partition Wall Damage; (b) Tensile Stresses Pattern from Analysis of 3D Model



(a) (b)

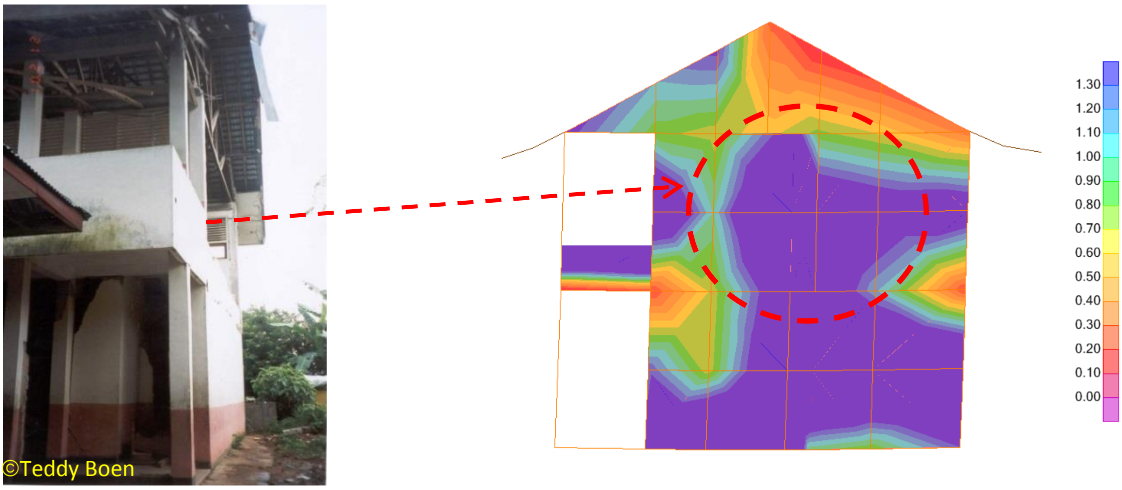
Figure 80 – (a) Actual Column Damage; (b) P-M Column Ratio from Analysis of 3D Model – Red Color Indicates Column Damage Possibility

4.5.3. Two-Stories Masonry School Building with Confinement

Figure 81 – 3D Model of Two Stories School Building with Confinement

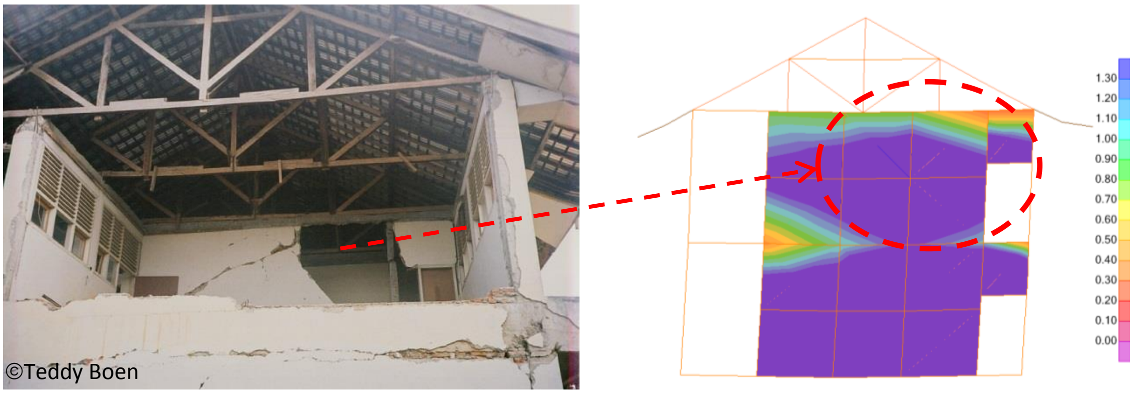
The analysis is elastic using response spectrum based on the Indonesian Seismic Code 2012 and ASCE7-10 Table 12.2-1 for Bengkulu area (Hard soil; PGA = 0.42g; Importance Factor = 1.5, R = 1.5).

Below are the results of 3D-analysis using SAP2000 v15. The blue color (▬) indicates the stresses exceed the permissible stress.



(a) *(b)*

Figure 82 – (a) Actual Gable Wall Damage; (b) Tensile Stresses Pattern from Analysis



(a) *(b)*

Figure 83 – (a) Actual Partition Wall Damage; (b) Tensile Stresses Pattern from Analysis of 3D Model

Chapter 5 Retrofitting Non-Engineered Constructions

5.1. Review Recent Retrofit Methods

Unreinforced Masonry (URM) can be strengthened using many methods (IAEE, 1980; Reinhorn, et al., 1985; Boen, 1992; Ehsani & Al-Saidy, 1997; ElGawady, et al., 2004; GangaRao, et al., 2007; United Nations Centre for Regional Development (UNCRD) & Center for Disaster Mitigation (CDM) ITB, 2007; Alam, et al., 2009; Plesu, et al., 2011; Ashraf, et al., 2011) i.e.

- Walls reinforcing overlays, jacketing:
    - In this method, independent reinforcing bars are applied surrounding the structural element and subsequently a self-supporting reinforced concrete cover or cement mortar is applied, forming a jacket.
    - Usually, this strengthening method is applied to columns where the lateral deformation and the compression is high.
    - If wall surfaces are strengthened with this method, both faces of the wall must be tied or anchored to each other. The jacketing overlay must be placed from the foundation by introducing a belt of reinforced concrete so that transfer of loads to the soil can be effective.
    - The reinforcement must be fixed to the masonry with steel connectors and staples. The skin facings / overlays are connected through mortar. Intervention with jacketing is applied on both faces with diffuse connections to get an effective result.
    - The aim is to improve the strength and stiffness and to obtain continuous confinement and to get a monolithic behavior.
- Shotcrete technique:
    - In this method, strengthening is done by spraying skin facings / overlays of a mix consisting of sand, cement and additives on the surface of a masonry wall lined with wire mesh as reinforcement.
    - The thickness of the shotcrete is made in accordance with the strength needed.
    - Wire mesh is used to minimize cracks in the shotcrete layer.
    - Shear stress on the shotcrete layer is transferred through anchors fixed in the masonry wall using cement grout or epoxy.
    - To get good bonding between masonry wall and shotcrete layer, it is advisable wetting the masonry surface prior to spraying the shotcrete.
    - The shotcrete layer does not affect cracking or ultimate load, it only affect extended inelastic deformation.

  - Generally, it is assumed that the shotcrete layer will resist all the lateral forces working on the retrofitted masonry wall. The bending and shear strength of the shotcrete layer is many times larger than the unreinforced masonry wall. Some cracks will occur in the masonry wall if the wire mesh past its yield point and the shotcrete strains is exceeded.
  - The main aim is increasing the load capacity and capacity to axial load of the retrofitted masonry wall.
- Rehabilitation using seismic bands technique:
  - In this method, strengthening is achieved by providing continuous reinforced concrete band or ring beam (also called collar beam) at different levels of the building. Such bands prevent out of plane failure of the walls.
  - Horizontal bands should provide as follows: lintel band incorporates in itself all door and window lintels; roof and floor band it is required where timber or steel floor/roof structure has been used; gable band; sill band just below the window openings.
  - The main aim is to prevent shrinkage, differential settlement cracks and enhance the seismic resistance of the building.
- Using Fiber Reinforced Polymer (FRP) composites:
  - FRP are nowadays widely used in industries such as construction, automotive, sporting goods, leisure, aerospace, etc.
  - In essence, the composite materials consist of resin matrix reinforced with carbon fibers, aramid fibers, glass fibers, etc.
  - FRP is different than conventional materials and usually consist of various layers of fibers with polymers, or bonding the layers of laminates.
  - To apply FRP technique to masonry walls, practically requires very little preparation works and because FRP is very light, there is no need to adjust the foundation design.
- Ferrocement technique:
  - In this type of strengthening, it is done by applying an isotropic composite material matrix based on high resistance cement mortar and single or multiple layers of steel meshes.
  - The tensile strength of the ferrocement depends on the nature of the mesh, the orientation and the thickness of the reinforcement.
  - The main aim of this method is to improve the behavior of the masonry walls to take up in-plane and out-of-plane loads. The wire mesh helps to confine the masonry units after cracking and thus improves in-plane inelastic deformation capacity and the ferrocement improves the out-of-plane loading and arching action.
  - Ferrocement is suitable for low-cost housing since the materials are mostly available in many places, and the workmanship is relatively easy and can be done by unskilled masons without any special tools.

From the above brief descriptions of retrofitting methods applied in practice, ferrocement is the ideal retrofitting method for developing countries like Indonesia since the construction is not too difficult and low cost. Therefore, this dissertation will be focused on strengthening of URM with ferrocement skin facings.

5.2. Ferrocement as Strengthening Layers in Sandwich Construction for Retrofitting Purpose

As explained in Chapter 5.1, ferrocement can be done with low-cost materials and by local masons which are readily available in developing countries. It is suitable to withstand earthquakes and wind loads if used to strengthen URM. Ferrocement is widely used as strengthening material, among others to strengthen reinforced concrete beams, columns, plates, etc. (Reinhorn, et al., 1985; Fahmy, et al., 2000; Amanat, et al., 2007; United Nations Centre for Regional Development (UNCRD) & Center for Disaster Mitigation (CDM) ITB, 2007; Shah, 2011). Apart from that, most importantly, ferrocement is used to strengthen URM, and many tests have been conducted on the subject matter, the tests demonstrated that ferrocement is able to increase the strength of URM. The mesh helps to confine the masonry units after cracking and thus improves in-plane inelastic deformation capacity. In a static cyclic test, this retrofitting technique increased the in-plane lateral resistance by a factor of 1.5. Regarding out-of plane behavior, ferrocement improves wall out-of-plane stability and arching action since it increases the wall height-to-thickness ratio. Results also showed that ferrocement overlay is a highly effective method of strengthening / repairing.

One of the experiments already conducted is for a concentrically loaded column damaged due to over-loading. The results showed that the strength of the column can be enhanced using a ferrocement jacketing. After repairing, all test specimens and analytical models showed higher deformation at ultimate load, increase in the ductility ratio, and considerable increase in energy absorption. The number of wire mesh layers in jacketing influenced the gain percentage of the ultimate load, ductility ratio, and energy absorption. The more reinforcement, the higher the gain for axial force, but the ductility ratio and energy absorption gain will be lower.

One other experiment was conducted for unreinforced brick masonry columns jacketed by ferrocement and the failure load becomes double. The test showed the following: that premature failure will occur if the wire mesh is not properly wrapped and plaster does not **FULLY** penetrate into it: that mortar strength has comparatively smaller influence on failure load. Mortar cover to wire mesh reinforcement shall not be greater than 2mm for ferrocement 6mm thickness and casing **ONE** layer of reinforcement may be satisfactory.

A shaking table experiment showed that brick-column masonry wrapped with wire mesh only, without mortar survived the shaking while the original brick-column masonry collapsed (Imai, 2008). If the wire mesh is embedded in mortar mix, the strength of the columns will no doubt be increased. Therefore, ferrocement using wire mesh is suitable for strengthening.

Another experimental investigation was done for a portal frame with infill masonry wall and subject to monotonic loading till failure. Subsequently the damage frame was repaired using ferrocement layers and tested again until failure. The result was that the failure load of the repaired frame is higher than the original frame. It was concluded that ferrocement overlay is a highly effective method of strengthening/repairing distressed reinforced concrete frame with masonry infill. The test also showed that the width of cracks developed in the repaired frame was smaller than those of the original frame. Based on this result, it was stated that if ferrocement overlay is applied to any existing undistressed infill, the lateral load capacity of the frame would significantly be increased. It was concluded that ferrocement has superior capability in protecting the repaired frame from environment.

If URM is strengthened using ferrocement, the walls will consist of three main layers: ferrocement, brick-wall, and ferrocement again. This structure becomes a "sandwich structure" with brick-wall as the core and ferrocement act as skin facings.

The instability failure modes in general for high-tech sandwich structures as explained in Chapter 1.3.2.3 are in general not present in the proposed retrofitting method to strengthen the URM using ferrocement skin facings. The core is the masonry wall, therefore it is solid and by itself as a structural member and the skin facings on two sides of the wall are tied to each other.

As explained in Chapter 3.3 page 68, bonding between the masonry wall and cement mortar plaster is excellent and the ties enhance the bond between masonry and the ferrocement layer and also to make local buckling in sandwich structures for the proposed retrofitting method unlikely. Only material failure is a possibility.

5.3. Proposed Retrofitting Method

The basic concept of seismic retrofitting is to improve the earthquake-resistance of structures without changing their existing basic framework. Retrofitting should be adopted only when the following provide the advantage over rebuilding:

- The material and construction cost can be limited to the minimum amount.
- The construction period can be limited to the minimum amount.
- The methodology should contain solutions to make those buildings earthquake resistant utilizing locally available materials and workmanship and suiting the local social, cultural, ethnographical, economical as well as political conditions.
- The methodology can be carried out by homeowners with minimal financial and technical assistance, and do not require extensive reconstruction or modification of the existing building.

Much can be learned from foreign research results and in principle some of the results could be applied provided they are adjusted to suit Indonesian conditions. For Indonesian non-engineered constructions, technological solutions wherein the common man can construct an ordinary earthquake-resistant house with locally available resources are needed. Such technology can be found by studying the site specific information and adhering to the local culture. In this regard, most probably foreign consultants must learn from Indonesia and not the reverse.

Methods of construction in Indonesia differ from methodogies used abroad, particularly with regard to non-engineered constructions. Therefore, several technical problems require indigenous research and development. There is a clear need to focus research on "engineering" of earthquakes as against the focus on "science" of earthquakes that many researchers have been doing.

In conjunction with the above principles, retrofitting with ferrocement is introduced. As mentioned above, ferrocement construction can be done by local labor under minimal supervision and the materials needed are relatively cheap, readily available in developing countries and the most suitable for Indonesia. Therefore, the author proposed to retrofit the non-engineered houses using ferrocement placed at both sides of walls. The analysis and design of this proposed method are analog with sandwich structures where the brick-wall acts as a core and ferrocement on both sides of the wall act as skin facings. The method for strengthening URM using wire mesh was introduced in Indonesia by the author in 1992 after the Flores earthquake (Boen, 1992, pp. 5-2). The method proposed consists of structural schemes which are safe, buildable, and affordable.

The idea of using ferrocement for strengthening was introduced in 1980 Monograph of Non-engineered constructions (IAEE, 1980) as can be seen in Figure 84. In fact, this is a sandwich panel with masonry wall as core and ferrocement as skin facings. However at that time no justification was given due to the fact that very few research works were available concerning sandwich panels as well as non-availability of analytical tools for the purpose. This technique also has been applied in many places, i.e. in Caribbean, after Corinth earthquake in Greece (Spence & Coburn, 1992).

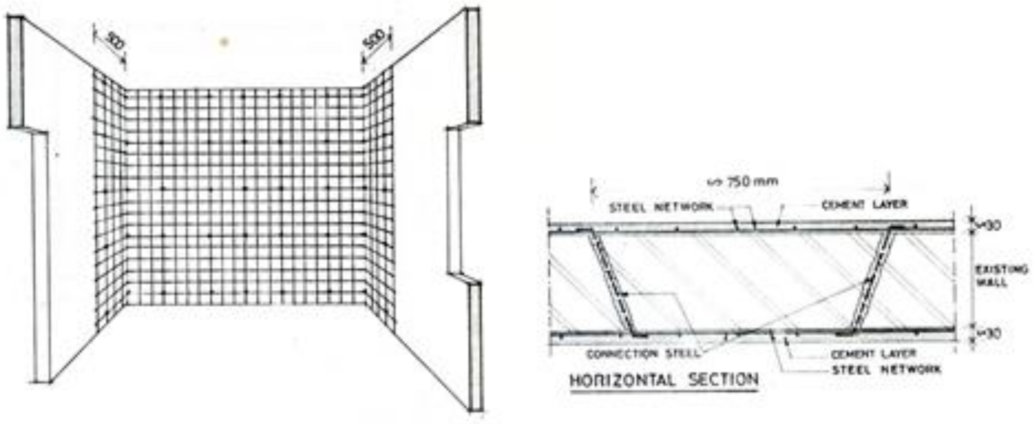


Figure 84 – Vertical Reinforced Concrete Covering Plates (IAEE, 1980)

As a variation and for economy in the use of materials, the covering may be in the form of vertical splints between openings and horizontal bandages over spandrel walls at suitable number of points only (IAEE, 1980). This method is applied as explained in Figure 89, p. 126 (Boen, et al., 2012).

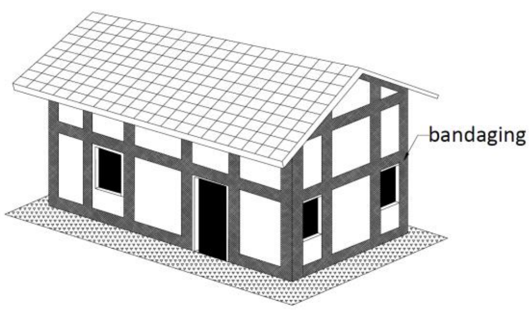


Figure 85 – Strengthening with Ferrocement Splints (IAEE, 1980)

Ferrocement has been widely studied and from the experiments results indicate the suitability of ferrocement as a retrofit material (Reinhorn, et al., 1985). Two failure mechanisms in ferrocement: a diagonal tension failure with ductile behavior, and a bond failure with brittle behavior. If one of the mechanisms developed, the other does not occur. The bond anchors between the masonry and the coating have a dominant effect on the development of these mechanisms. The strength, ductility and secant stiffness degradation of the coated walls have values nearly double those for an uncoated wall and the composite strength does not appear to depend on mesh size.

Many papers that have been published mentioning that the walls were strengthened using ferrocement; however, the wire mesh was placed right next to the brick-wall (see Figure 86). This is contrary to the principle of ferrocement which will be described in the next section. The proposed retrofitting method is strengthening brick-walls using ferrocement on both sides, where the wire mesh is embedded in mortar forming a ferrocement layer, and the wire mesh is not directly attached to brick-walls.

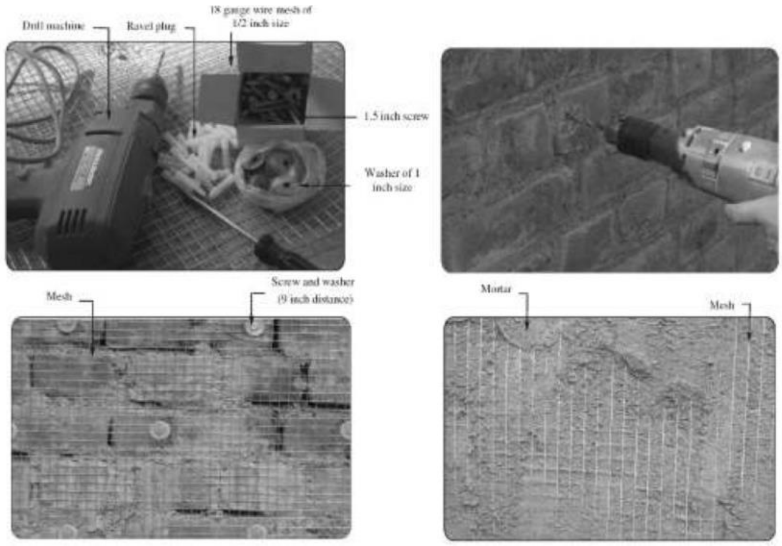


Figure 86 – Example of "Ferrocement" Strengthening, but the Wire Mesh was Placed Right Next to the Brick-Wall (Ahmad, et al., 2012)

There are few researchers that mentioned one of the retrofitting methods of walls using ferrocement with welded mesh located at the center of the ferrocement layer (see Figure 87). Almost all papers did not explain in detail how to implement it and also did not explain how to analyze the wall as a sandwich structure, in which brick-walls act as a core and ferrocement on both sides of the walls act as a skin facing. Another paper deals with similar strengthening URM, also using ferrocement, however, sandwich analogy was not applied (ElGawady, et al., 2004; Muntean, et al., 2010). The wire mesh was applied on both sides of walls as detailed in Figure 87. The wire mesh was directly in contact with the brick-walls.

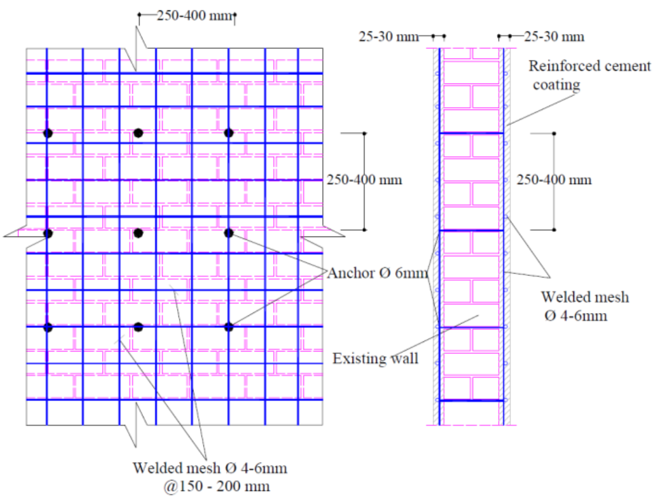


Figure 87 – Example of Strengthening Walls using Ferrocement (ElGawady, et al., 2004)

The method applied and introduced by the author (Boen, 2010). First ±1cm thick, ±2cm width, with spacing ± 10cm plaster is made to serve as support for the wire mesh. Such thin plasters "supports" can be replaced by umbrella head roofing nails which serve as wire mesh supports and at the same time serves as shear connectors to strengthen the bond between mortar and the URM (see Figure 88). The "supports" are necessary so that the wire mesh is fully encapsulated in the mortar matrix and thus becomes ferrocement layer.

Steel welded wire mesh, made with minimum 1 mm diameter wires spaced at maximum 25mm in both directions, was stitched to each other using several strands of tie wire fixed in 10mm holes drilled in the mortar layer. The distance between strands is approximately 40-50cm. A minimum of 15cm wire mesh overlapping width, in vertical and horizontal direction were provided at connections of two wire mesh sheets. The average tensile strength of wire was ±6670 kg/cm^2 (Testing Research Center and Residential Development Laboratory (Puskim), 2013).

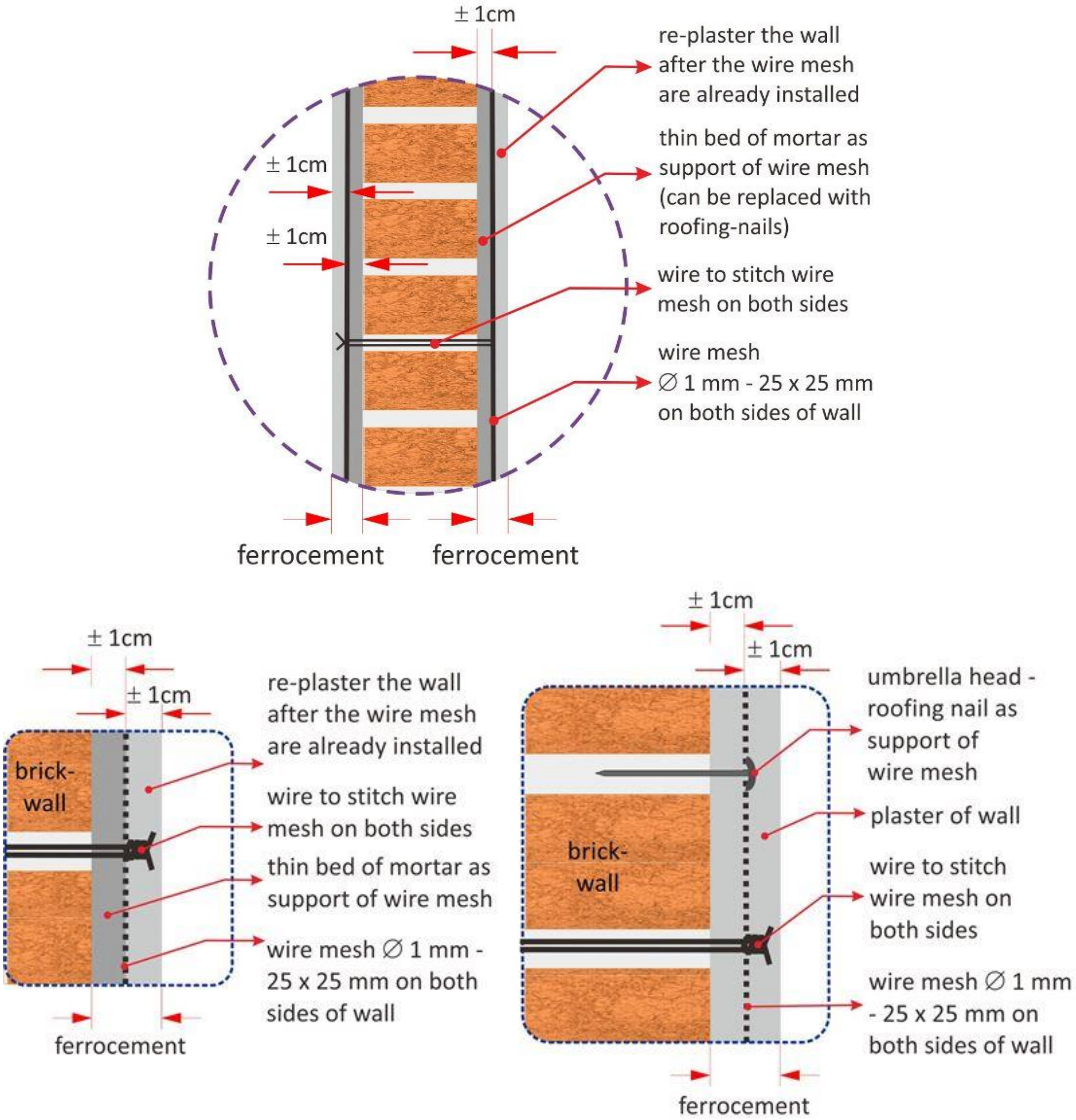
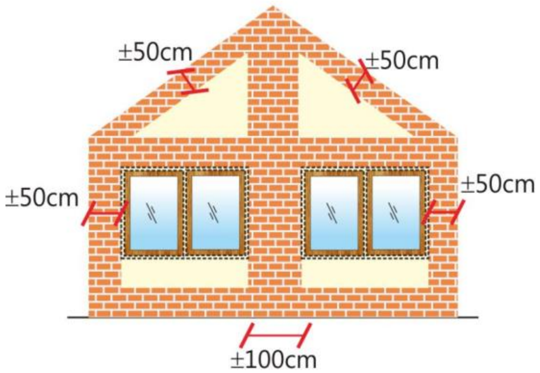


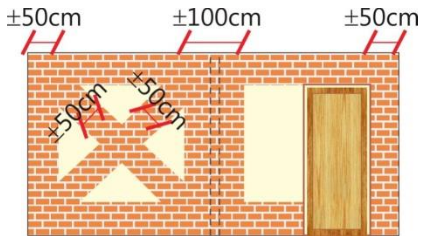
Figure 88 – Detail of Reinforcement using Wire Mesh (Boen, 2010)

The procedures of retrofitting a damaged building using ferrocement are as follows (Boen, 2010):

1. Remove the plaster layer around 30-50 cm width vertically and horizontally along corners of the wall and at the corner of openings Figure 89(a) on both inside and outside building. For wall without opening, remove the plaster layer diagonally Figure 89(b&c) on inside and outside as well. If the existing mortar is fragile, peel the whole building plaster layer Figure 89(d).



(a)



(b)

(c)

(d)

Figure 89 – Remove Plaster Layer on both Inside and Outside Building

2. The cracks are sealed with cement and sand mortar (Figure 90). Spray the walls with water if the mortar consists of sand and cement. However, if the mortar of the existing walls consist of lime and sand or lime, red-brick powder and sand, without cement, it is better to inject cement water into the mortar.

Figure 90 – Cracks are sealed with Cement - Sand Mortar; Walls are sprayed with Cement-Water

3. Make supports to place the wire mesh so that the wire mesh is not directly in contact with the brick-walls. There are two alternatives to make supports. First is by making a thin bed mortar, and second is by using umbrella-head-roofing-nails. The last alternative is more simple and faster than the first one. However, if the project is located in the isolated area and it is difficult to find the umbrella-head-roofing-nail, the first alternative is more reliable.

   Alternative I: Supports using thin bed of mortar

   a. Thin bed mortar with 10mm thick, ±2cm width, and spacing ±10cm must be made that serves as the support for the wire mesh. Use 9mm thick and ±2cm width plywood strips as formwork.
      i. Prepare 2 pieces of plywood place as shown in Figure 91(i).
      ii. Place mortar among two plywoods (Figure 91(ii)).
      iii. After the mortar harden, remove the plywoods and move to the next ±10cm spacing (Figure 91(iii)). If the bed thin mortar width is more than 2 cm, the space between each thin bed can be more than 10 cm.

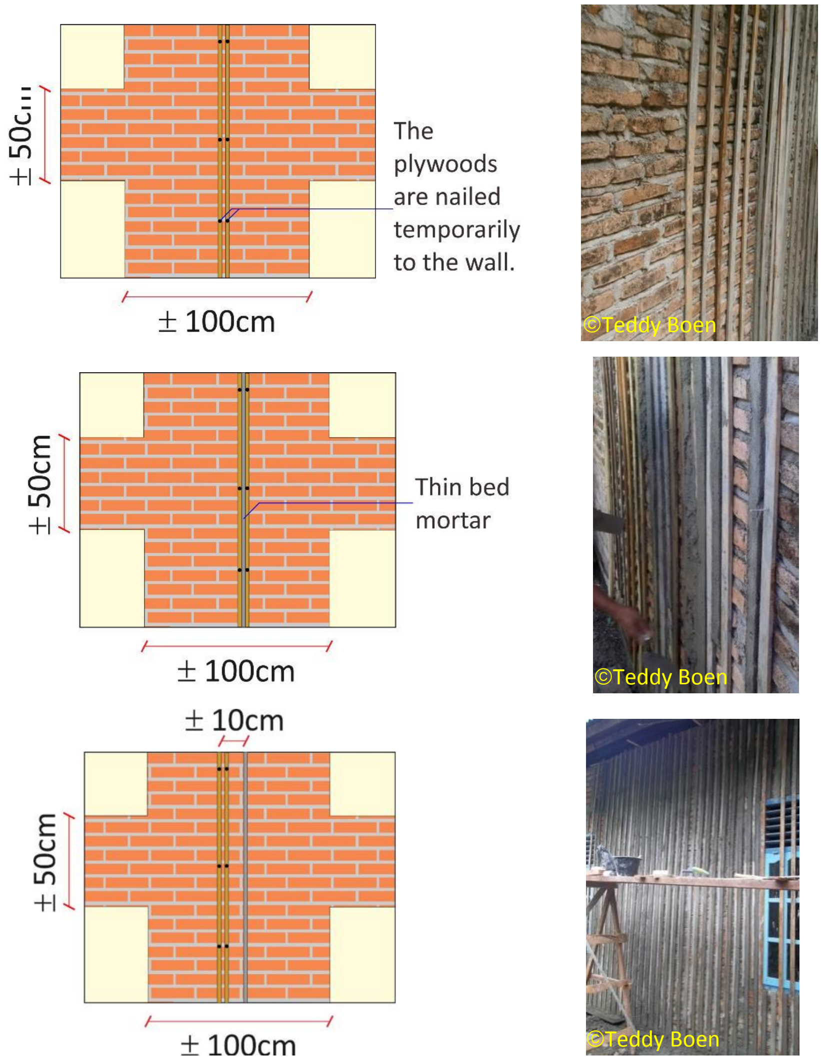


Figure 91 – Process of Making Thin Bed Mortar

   b. Place the wire mesh at both sides of the wall; nail the wire mesh to the thin beds mortar in several places of the wall to keep the wire mesh in place and does not shift (Figure 92). Give an extra overlap 15 cm at the wire mesh connections.

Figure 92 – Installation of Wire Mesh; Use Nails to Keep the Wire Mesh in Place

Alternative II: Supports using umbrella-head-roofing-nails

a. Nail the umbrella-head-roofing-nails at the corners of the wall, at the corner of openings and at diagonal area where the plaster is already peeled both inside and outside (Figure 93). The spacing between each nail is approximately 20 cm and 1 (one) cm from the wall surface. This can be easily done using 1 cm thick plywood as guide. The roofing-nails also act as shear connectors.

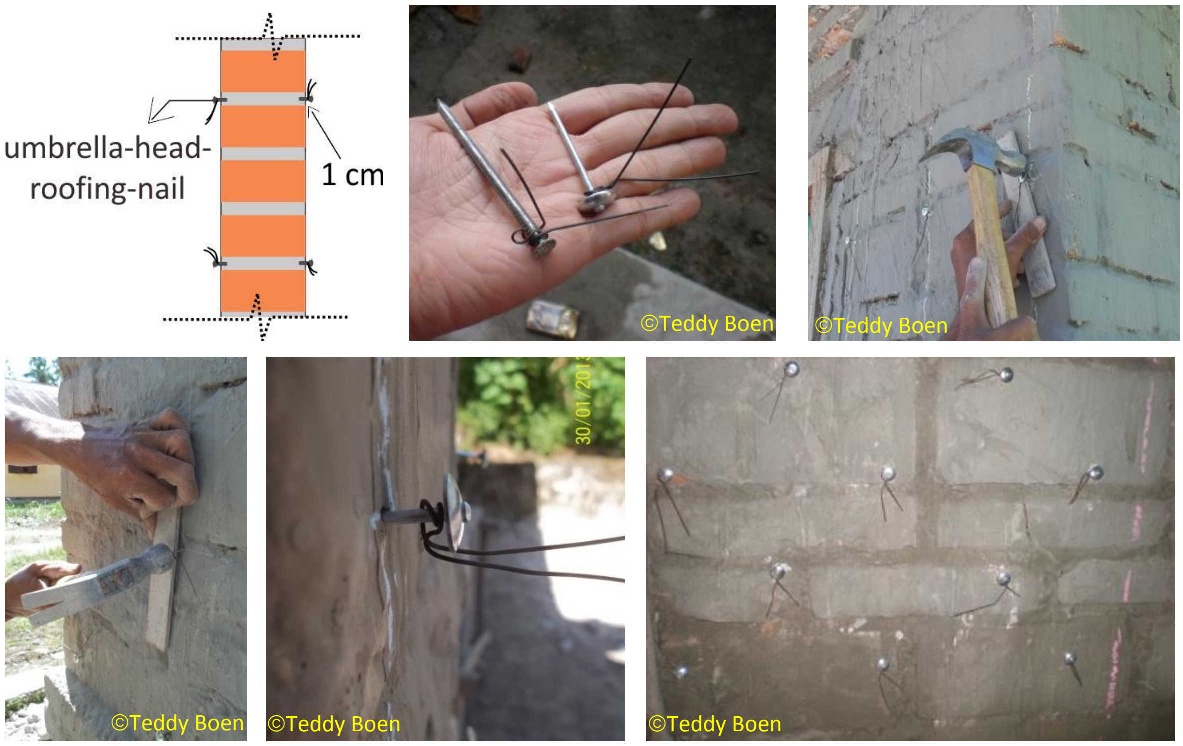
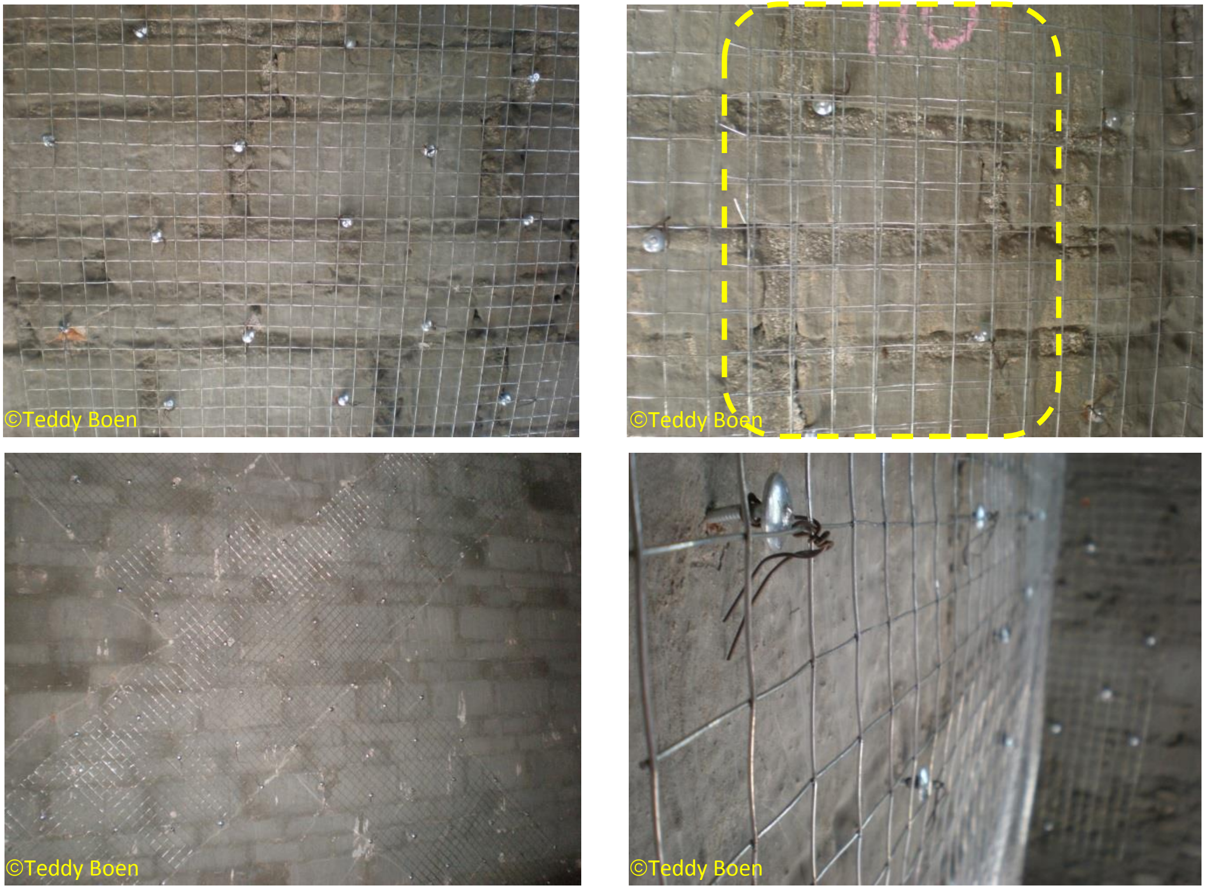


Figure 93 – Installation Umbrella-Head-Roofing-Nails as Supports of Wire Mesh

b. Place the wire mesh and tie to the umbrella-head-roofing-nails using tie wire at both sides of wall (Figure 94). Give an extra overlap 15 cm in the wire mesh connection.



(a) *(b)*

Figure 94 – (a) Installation of Diagonal Wire Mesh; (b) Wire Mesh Installed and Fastened to Top of Umbrella-Head-Roofing-Nail

4. Make holes by drilling the masonry wall at the mortar layer, with spacing of approximately every 40 cm to stitch the wire mesh on the inner side of the masonry wall to that of the outer side (Figure 95). If the support is using thin bed mortar, the hole must be drilled on those thin bed mortar and not on the brick-wall.

Figure 95 – Drilling the Walls for Stitching

5. Insert tie wire into the holes and stitch the inside and outside wire mesh so that the ferrocement layers on two sides of the wall are tied to each other (Figure 96). The

ties are meant to strengthen bonding and also to make local buckling in the sandwich structures is unlikely.

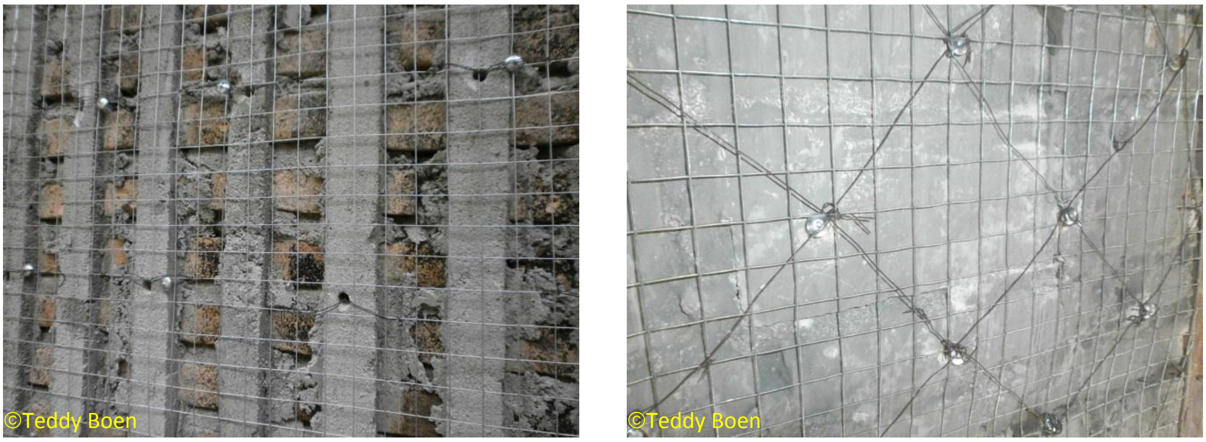


Figure 96 – Stitch the Inside and Outside Wire Mesh to Prevent Local Bucking

6. Grout the hole using cement paste as can be seen in Figure 97.

Figure 97 – Grouting of Drill Holes Using Cement Paste

7. Plaster the wall using 1 PC : 4 sand mortar with thickness 1cm on top of the wire mesh (Figure 98). Therefore the ferrocement layer is 2 cm thick with the wire mesh encapsulated in the middle.

Figure 98 – Re-plaster the Walls; Wire Mesh in the Middle of the Ferrocement Layers

Such retrofitting method using ferrocement as strengthening layers was used when retrofitting two school buildings in Bandung, i.e. SDN Cirateun Kulon II in 2006 and SDN

Padasuka II Soreang in 2007 (United Nations Centre for Regional Development (UNCRD) & Center for Disaster Mitigation (CDM) ITB, 2007). The projects were a collaborative project to reduce the vulnerability of those existing school buildings in the corridor of School Earthquake Safety Initiative (SESI) project. After retrofitting, those school buildings survived the September 2, 2009, West Java earthquake without significant cracks.

Many retrofitting projects were also done using this proposed retrofitting method, especially after the September 30, 2009 West Sumatra earthquake, many school buildings and also an engineered building were retrofitted using this method.

SDN Padasuka II Soreang

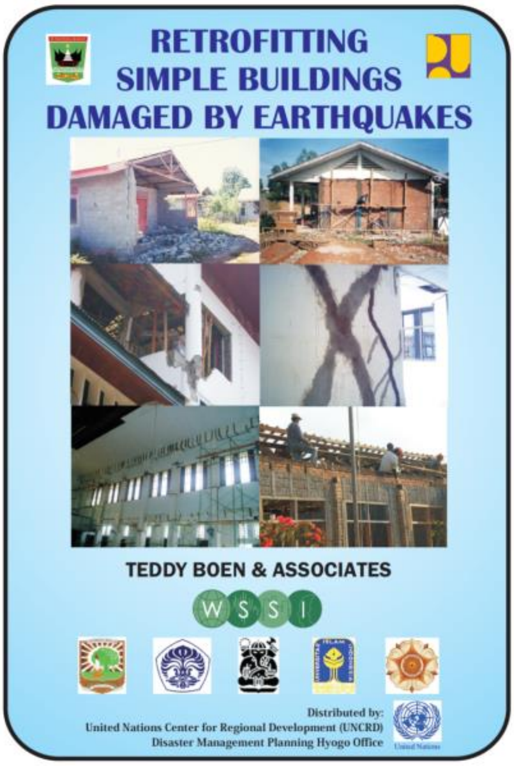
SDN 13 Batu Gadang, Padang

Bumiminang Hotel, Padang

Figure 99 – Example of Retrofitting Buildings using Wire Mesh

This proposed retrofitting method has been published as guidelines for retrofitting of buildings in Indonesia (see Figure 100):

- 2009, Retrofitting Simple Buildings Damaged by Earthquakes, Boen, et.al.
- 2012, Buku Panduan Perbaikan dan Perkuatan Bangunan Tembokan Sederhana, Boen, et.al.



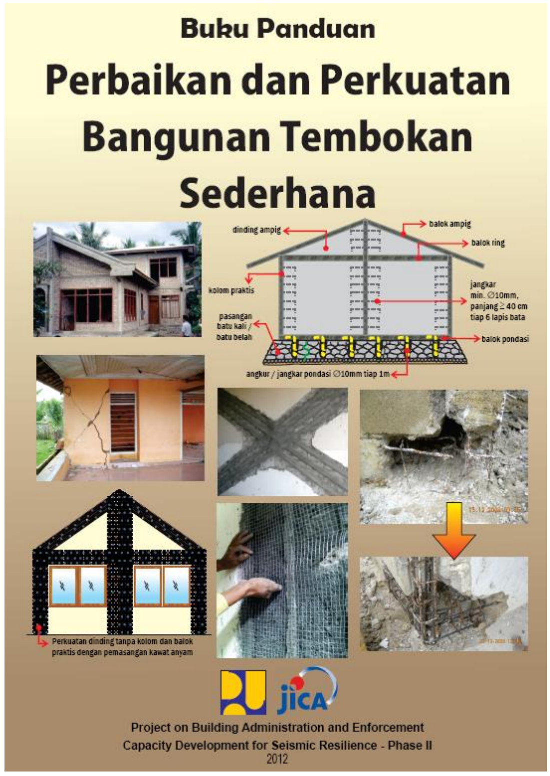


Figure 100 – Guidelines for Retrofitting Simple Buildings in Indonesia

From experience in retrofitting a house in Padang, West Sumatra that was completed in 2012 in cooperation with JICA, the cost to retrofit using this proposed method is much smaller than to build a new house. The retrofitting cost should be 15-20% of the cost to build a new house (see Chapter 6.1.1 and 6.1.2). Therefore this proposed method is feasible to be applied in developing countries where the amount of resources required for retrofitting is very limited.

5.4. Characteristics of Masonry Walls Needed to Implement Sandwich Structures Principles

Many experts considered masonry as a non-elastic, non-homogenous and anisotropic material and therefore displays distinct directional properties in the masonry units and mortar. The masonry units and mortar are considered as planes of weakness (Bosiljkov, et al., 2005; Haadi & Dilrukshi, 2009; Gesualdo & Monaco, 2011).

Based on those deliberations, masonry walls has different properties along different axes based on the units texture, directions and geometry of the mortar joints and as a result, the mechanical properties of masonry depend on the properties of the composite components and on the interaction of the of composite components as well. The composite components consist of brick, mortar, their bond properties, their volume ratio, cracks in the masonry, and the orientation of the bed joints. The values of shear modulus and the stiffness of masonry elements depend on several factors. Almost all the parameters needed to define those characteristics cannot be measured on site. The parameters that can be measured are the elasticity modulus, the Poisson's ratio for the direction parallel to the bed joints and the stress on the external face of walls when cracks start to develop.

5.4.1. Characteristics of Mortar

Based on (Bosiljkov, et al., 2005), historic mortar is weak because the mortar consists mostly of lime mortar. Therefore the joints are weak and the general isotropic behavior assumption in creating a model will give different results from actual measured behavior. Masonry that is weak in the joints cracks along the weakest parts rather along the principle stresses and this is not in accordance with isotropic assumption.

The basic mortars used were a cement mortar (CM), a cement lime mortar (CLM), and a lime mortar (LM). Most of the unreinforced masonry built in accordance to Dutch tradition as mentioned in Chapter 3.2.2 are using lime mortar. However, almost all masonry "non-engineered" buildings that are built after Indonesia becomes an independent nation are using cement mortar. The cement mortar (CM) represents an assumed nominal isotropic material typical of modern construction where the mortar is relatively strong; the lime mortar represents a soft historic mortar with nominal anisotropic properties, and the cement lime mortar represents a typical contemporary mortar used extensively in the last five decades.

The results show that the effective stiffness of masonry elements depends on the types of mortar, the types of masonry, and levels of pre-compression. The effective stiffness and the overall resistance of the walls depend on the types of masonry, however, different types of masonry showed similar failure mechanism when subjected to the same pre-compression.

Table 6 – Elasticity Modulus and Shear Modulus of Various Mortars (Bosiljkov, et al., 2005)

Mortar Type	CM	CLM	LM	CLMR
Elasticity modulus (GPa)	12.6 ± 0.75	12.5 ± 1.2	1.8 ± 0.2	10.3 ± 1.1
Coefficient of variation (%)	6	10	12	11
Poisson's ratio	0.07 ± 0.03	0.25 ± 0.03	0.4 ± 0.3	0.28 ± 0.11
Coefficient of variation (%)	43	12	78	41
Shear modulus (GPa)	5.9	5.0	0.64	4.0

Depending on the level of accuracy and simplicity desired, different modeling strategies have been used by masonry researchers and can be explained as detailed micro-modeling, simplified micro-modeling, and macro modeling (Figure 101) (Kormanikova, 2003; Elgwady, et al., 2002; Ghiassi, et al., 2008; Haadi & Dilrukshi, 2009; Luccioni & Rougier, 2012).

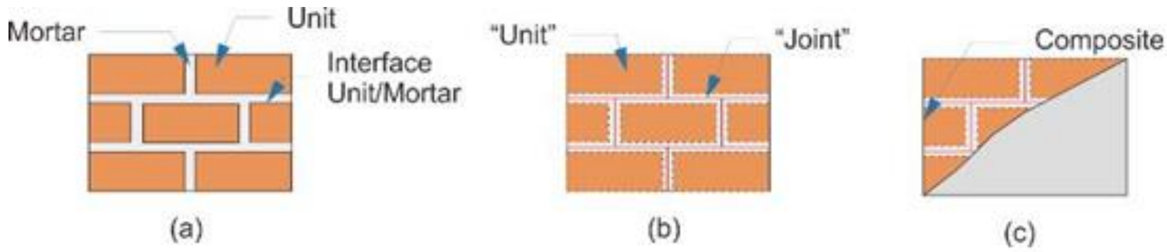


Figure 101 – Modeling Strategies for Masonry Structures: (a) Detailed Micro-modeling; (b) Simplified Micro-modeling; (c) Macro-modeling

5.4.2. Micro Modeling of Masonry Walls

Micro modeling represents brick, mortar, and brick-mortar interface separately and the elasticity modulus, the Poisson coefficient and if the analysis is nonlinear, the inelastic properties are taken into account. Masonry units and mortar are represented by continuum elements while unit-mortar interface is represented by discontinuous elements. Micro modeling analysis takes a lot of time and this approach is not suitable for analyzing actual structures. To develop a micro model is not easy since the model must take in to account the correct complex behavior of the brick and the mortar. Therefore, it will be very difficult to do a non-linear analysis for masonry walls because the material behavior is anisotropic and not homogenous.

Theoretically, micro modeling is more accurate than macro modeling, however, it takes longer computational time and cost and in most cases due to the high number of degrees of freedom, and its applicability is very limited. Micro modeling is generally used for research to get an understanding of the local behavior of masonry structures and can predict better results when structural and geometric features become complicated. In actuality, development of stresses can be caused by external loads, movement of one part of the structure, change of the chemical action due to moisture, and micro modeling is restricted to small test problem due to the many variables and data involved. In real practice, the analysis does not need to know the interaction between units and mortar which is negligible for the overall structure behavior. For actual structures, macro model technique is commonly used and the size of the masonry structure is assumed larger than the bricks and mortar joints.

According to (Bosiljkov, et al., 2005), soft-lime mortar is considered an anisotropic masonry material and the behavior during earthquakes is entirely different compared to hard-brittle mortar. From shear tests of masonry walls, the masonry shear modulus calculated from the effective stiffness vary from 6% to 25% of the measured elastic modulus of the masonry. The shear modulus is an intrinsic property of a material and the results should not depend on the test method.

5.4.3. Macro Modeling of Masonry Walls

Due to numerous uncertainties in the brick wall; such as the quality of brick, mortar's strength, workmanship, non-uniformity of mortar thickness in the joints, curing of the test piece, test procedure, method of loading and handling of specimen, etc. a quantitative analysis especially applying very sophisticated methods of analysis is considered as a waste of time and energy as can be seen in Figure 102. Application of simple algebra and strength of materials formula are more than enough for such analysis. In some cases engineering judgments and qualitative analysis will be quite appropriate. It should not be a brain teasing problem, however, must be practical and applicable solution. The intent is to identify things that add complexity to the design process and try to work to simplify them. It is easy to make something unnecessarily complicated. It requires a lot more work to make it simple.

As explained earlier, in macro modeling there is not any distinction between brick, mortar and brick-mortar interface. To simplify the analysis and design, the masonry is assumed as a linear elastic, homogeneous and isotropic material and the forces, stresses and strains are usually determined on the gross cross-section of the walls (Mosalam, et al., 2009; Gesualdo & Monaco, 2011; Ghiassi, et al., 2008; Churilov, 2012). Nonlinearity only occurs when masonry is under compression-compression state of stress. Initially, masonry behaves as a linear elastic material; therefore, masonry can be modeled, on average, as an isotropic continuum (Mosalam, et al., 2009). Current code procedures also assume the behavior of the masonry as an isotropic material (Bosiljkov, et al., t.thn.). The advantages of this assumption are that the simplest theory of elasticity provides the analysis procedures. Modeling and analyzing isotropic assumption is not too complicated and is acceptable for modeling an actual building. As a different technology, mortar may also be regarded as masonry, not as being composed of small elements, but cast into a continuous structure which hardens chemically and becomes one with the bricks forming the masonry wall (Heyman, 1999).

Based on past study to investigate the behavior of the masonry wall using macro modeling approach, in general, for the load-bearing walls resulting stress patterns of corresponding isotropic and anisotropic models are similar (Haadi & Dilrukshi, 2009).

5.5. Application of Sandwich Structures Principles to Strengthen the Unreinforced Masonry (URM)

Even though sandwich structures are commonly used in modern high-technology application with weaker core and metal skin facings, the principles of sandwich structures can be applied to strengthen non-engineered masonry buildings, with URM as core and ferrocement as skin facings. Unlike common sandwich structures, masonry core is quite strong and can be used as structural member (Baker, et al., 1972).

Figure 102 – Poor Workmanship Non-uniformities in Actual Brick Walls

Figure 102 (cont'd) – Poor Workmanship Non-uniformities in Actual Brick Walls

Based on observations for the past 40 years, surveys and tests which have been conducted by various agencies as mentioned in Chapter 3.6.1, in Indonesia, there are many variation of bricks dimension. The quality of brick-works also vary, from good enough until poorly brick-work, as can be seen in Figure 102.

With these facts, it would not be appropriate and a waste time and energy if the masonry walls are analyzed using complicated theory as micro modeling (see Figure 103). There is a need for uniformity of masonry materials and workmanship. Quality control is a necessary feature for any construction material and system to achieve confidence and credibility. As explained earlier, for Indonesian conditions, quality control masonry construction is very difficult to maintain in actual condition. Therefore, the masonry walls should be modeled and analyzed using macro modeling principles, which is assumed as linear elastic, homogeneous and isotropic material.

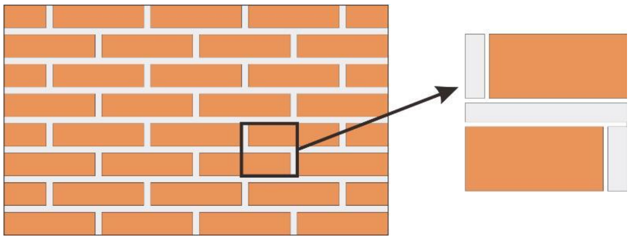


Figure 103 – Modeling Masonry Wall using Micro-Modeling Concept is Unnecessary

Sandwich Structures Design for URM as Core and Ferrocement as Skin Facings

Multi-layer construction has become more and more important in structural engineering as a mean for achieving a beneficial combination of the properties of two or more materials (Abel & Popov, 1968). The best known examples of this type are the widespread "sandwich" structures used in the aerospace industry. These combine thin, high-strength facing layers with a thicker, light-weight core.

The theory of stress analysis of multi-layer structures is well established (Allen, 1969; Baker, et al., 1972). In general, there are two types, namely "sandwich" and "laminates". In sandwich structures, some layers are weaker than others and transverse shear deformation is taken into account and in "laminates", the layers of materials with similar properties and thickness are bonded together and for the analysis, the Kirchhoff-Love hypothesis is used.

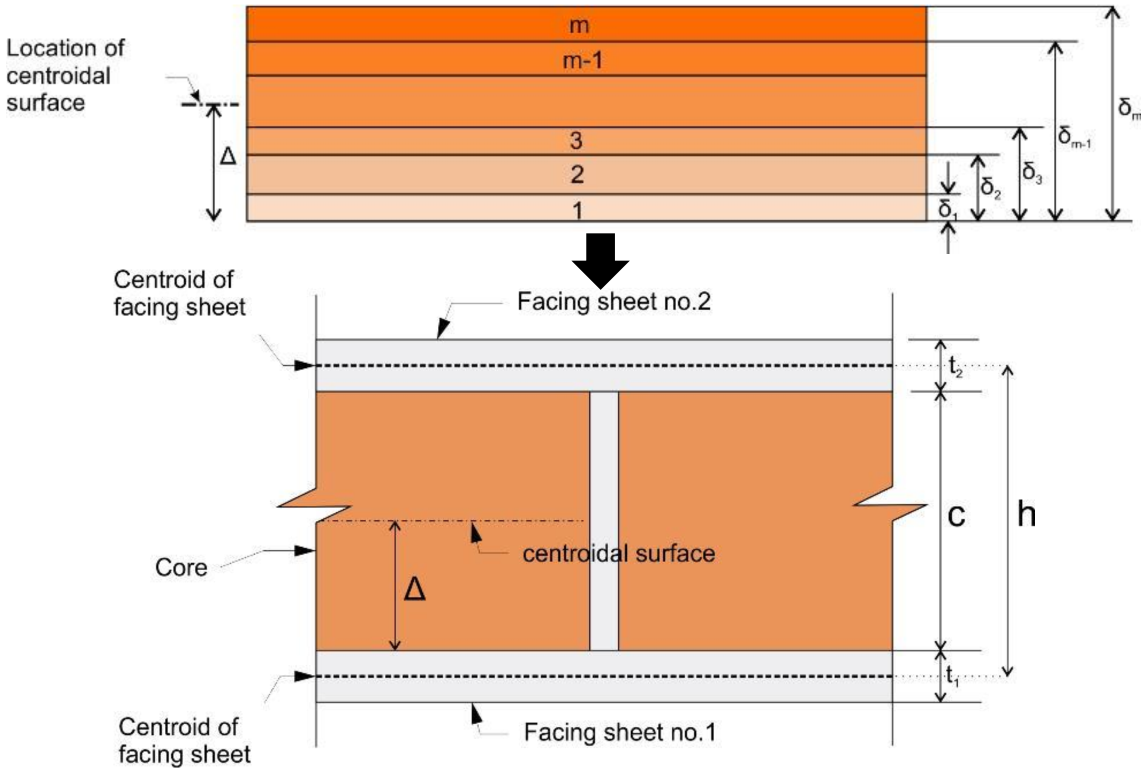


Figure 104 – Multi-Layered Construction (top) and Sandwich Structures (bottom)

As explained earlier, sandwich structures analysis and design originated from multi-layered shells composite formed by bonding several layers. Sandwich structure is a layered composite formed by bonding two thin faces to a thick core. Therefore, the analysis and design equations originated from multi-layered shells composite. A typical multi-layered cross section is shown in Figure 104.

A conventional analytical method for sandwich structures is elaborated in (Allen, 1969; Baker, et al., 1972). Conventional method involves finding mathematical equations which define the required variables and subsequently finding the solutions. In many cases practical problems become extremely difficult to solve. Most of the references are directed toward the structural analysis rather than the design, therefore, very few has been expended toward possible simplified equations or design tables, charts, that can be useful for design. Moreover, these solutions have been restricted to the simpler geometries such as rectangular and circular plates and cylindrical and spherical shells.

To perform analysis and design for seismic evaluation and strengthening of masonry structures using computers, a model needs to be created. It is difficult to solve by means of a slide rule or hand calculator (Hartsock, 1969). The equations derived are complicated and are not easy to solve manually. The presence of a vast range of geometrical and structural configurations of masonry, physical models to investigate masonry is costly and difficult. As a result finite element method (FEM) has been widely used in the analysis of masonry structures.

With the advancement of high-speed computers, sandwich structures analysis and design has moved toward more versatile numerical method, namely Finite Element Method (FEM). This is elaborated in (Hughes, 2000; Abel & Popov, 1968). No mathematical model of a structure can completely reproduce the actual behavior (Meyers, 1983). Much depends on the structural engineers that make the models. Differences can occur due to the following: determining approximations that are suitable for the particular structure and checking whether those approximations are valid. Last but not least, to make sure that after designing the behavior of the structure is in line with the model used created for the analysis. If the analysis model captures the behavior of the actual structure more closely, the analysis results are more likely to be reliable.

Sandwich structures can be analyzed using commercial softwares SAP2000 / ETABS 2013 and utilizing layered shell based on FEM. In the manual of SAP2000 / ETABS 2013, the layered shell allows any number of layers with different thicknesses as well as properties to be defined in the thickness direction, each with an independent location, thickness, behavior, and material (CSI Berkeley, 2013). Material behavior may be linear or nonlinear. The material behavior is integrated (sampled) at a finite number of points in the thickness direction of each layer and the location of these points follows standard Gauss integration procedures (integration points).

Membrane deformation within each layer uses a strain-projection method (Hughes, 2000). In-plane and out-of-plane displacements are quadratic. Mindlin/Reissner formulation is used for bending which always includes transverse shear deformations. Layers are kinematically connected by the Mindlin/Reissner assumption that normal to the reference surface remain straight after deformation, means that the transverse shear strain is the same in every layer. This is the shell equivalent to the beam assumption that plane sections remain plane.

5.6. Experimental Tests of Brick-wall Panels Strengthened using Wire Mesh

Actually, many experiments using samples of brick-wall segments were conducted to find out the cracking behavior and the compressive strength of un-retrofitted and retrofitted brick masonry walls. The strengthened wall acting as a composite material will have the cracking behavior and failure modes corresponding to its properties which may be different from the unreinforced masonry (Ghiassi, et al., 2012).

If a masonry wall is strengthened by lining with wire mesh layers anchored to the wall, the behavior of the composite wall will be different than unreinforced masonry wall. This is because of the substantial capacity of the mortar layer and wire mesh reinforcing.

In URM, cracks are substantial and concentrated in small areas in the wall and in strengthened masonry walls due to bonding of the wire mesh, the cracks are distributed more evenly. Different strengthening details will also cause different non-linear behavior and result in different failure modes. This is not considered in conventional design.

A masonry wall with rocking failure mode, if already strengthened can change to diagonal tension and this change affects the ductility and strength of the wall. If such change of failure mode is not considered in the design and evaluation procedures, the retrofitted becomes inaccurate (Ghiassi, et al., 2012). As explained earlier, ferrocement skin facings on masonry walls increases the compressive as well as tensile of the walls.

Ferrocement specimens having one layer of wire mesh wrapped around showed an increase in failure load of up to 40% as compared to controlled specimen (Shahzada, et al., 2012; Ahmad, et al., 2012).

5.7. Experimental Test of Wire Mesh Tensile Strength

To verify the tensile strength of wire mesh that was used in ferrocement layer proposed for strengthening, the author tested the wire mesh in Bandung. The test was conducted at the Testing Research Center and Residential Development Laboratory based on ACI 549.1R-93 (ACI Committee 549, 1999). The result showed that the average tensile strength of wire mesh is about 6770 kg/cm^2.

Table 7 – Wire Mesh Tensile Strength Test Results

No	Code	Dimension (mm)		Unit area (cm^2)	Yield force (kg)	Max force (kg)	Yield Stress (kg/cm^2)		Tensile Stress (kg/cm^2)	
		Ø	P					Average		Average
1	KT-G-A	0.75	600	0.0221	140.67	193.68	6368.36	6768.31	8768.03	8337.32
2	KT-G-B	0.75	600	0.0221	145.77	180.43	6599.10		8168.12	
3	KT-G-C	0.75	600	0.0221	162.08	178.39	7337.46		8075.82	

(a) *(b)* *(c)*

Figure 105 – Test of Wire Mesh Tensile Strength; (a) Before Test; (b) After Test; (c) Fractures of Three Specimens after Pull-Out

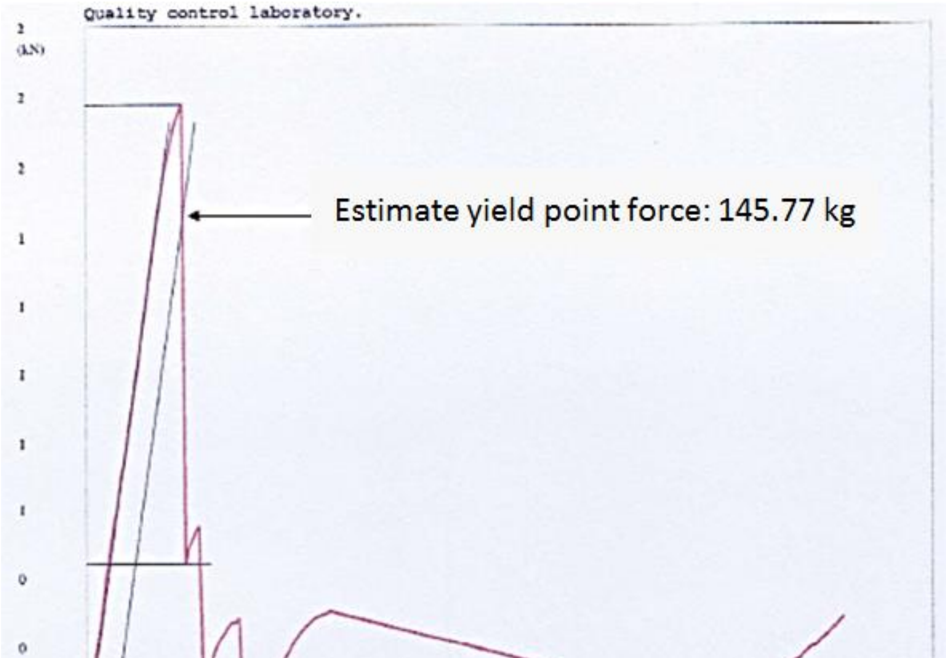
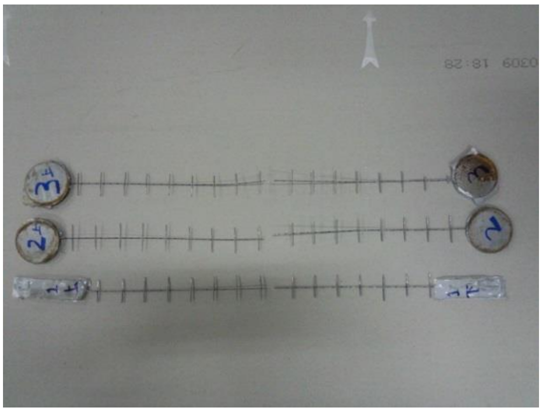


Figure 106 – Force-Deformation Graph of Wire Mesh Tensile Test

Another test of wire mesh tensile strength was also conducted in Japan (Imai & Nakatani, 2012) using the Indonesian wire mesh. The average tensile strength is about 7671 kg/cm^2. Therefore, this high tensile strength of wire mesh can absorb the tensile force occurred in walls due to earthquakes.



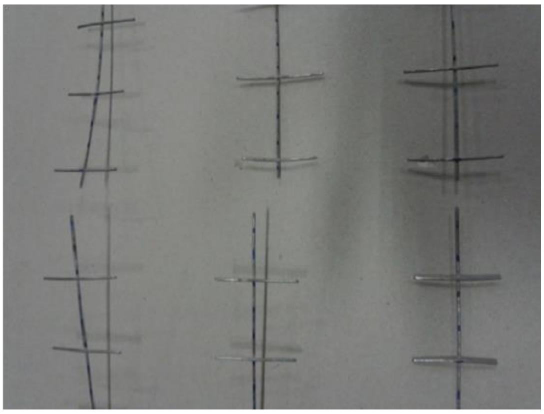


Figure 107 – Three Specimens of Wire Mesh after Tensile Test Conducted in Japan (Imai & Nakatani, 2012)

5.8. Shaking Table Test

5.8.1. Experiment of Masonry Columns Wrapped with Wire Mesh

The experiment was done by Science and Disaster Prevention Research Institute, Mie University in January 14, 2011. The purpose is to observe the behavior of the proposed method of earthquake retrofitting masonry house using wire mesh (Imai, 2011).

5.8.1.1. Structure Model

4 specimens of brick masonry columns were prepared (Figure 108):

- Model A: Unreinforced brick masonry column
- Model B: Brick masonry column reinforced using one ∅10mm rebar in the middle
- Model C: Brick masonry column wrapped by chicken wire mesh
- Model D: Brick masonry column wrapped by wire mesh similar with Indonesian wire mesh ∅1mm, 25x25mm.

5.8.1.2. Material Properties

The experiment was conducted on December 28, 2010 using Japanese bricks 60x110x230mm dimension. The brick mortar joint is 1PC : 6 sand with 15mm thickness, moisture content of 6.87%, and rate of water absorption is 1.74%. The height of the columnar body masonry is about 2000mm.

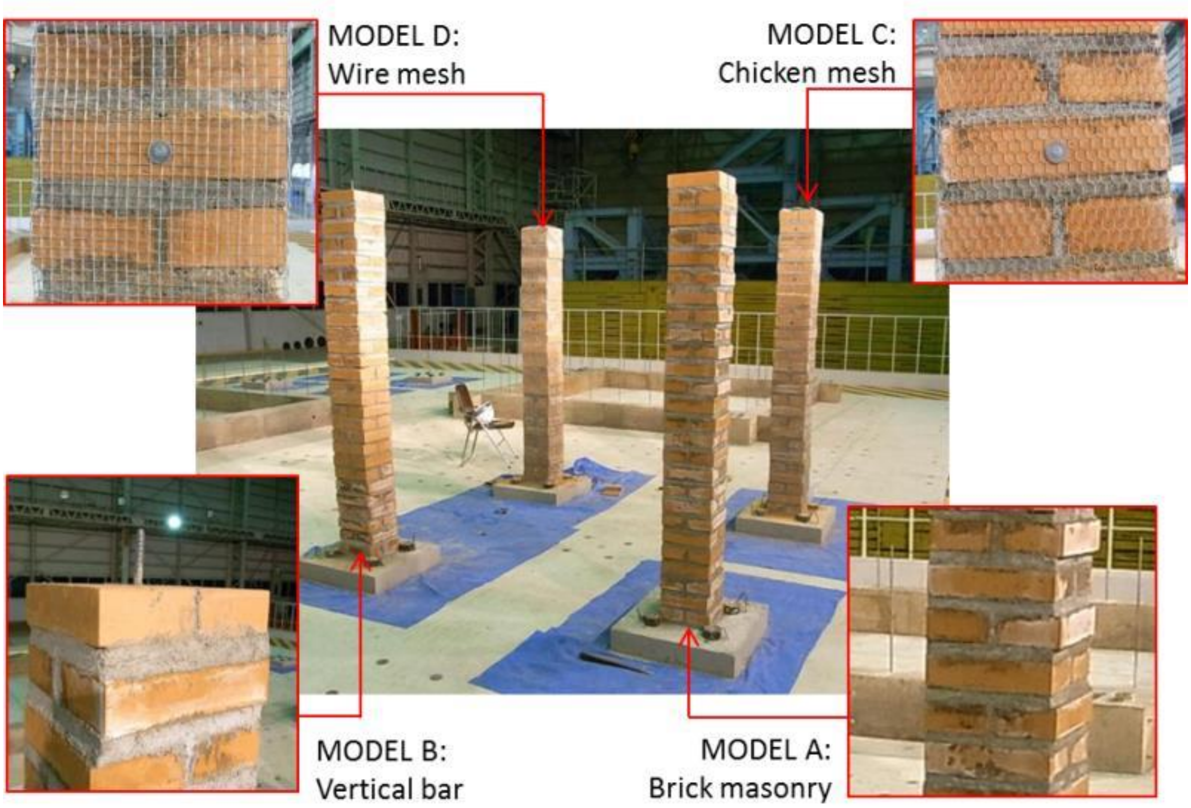


Figure 108 – Brick Masonry Column Model (Imai, 2011)

5.8.1.3. Input Motions

The excitation can be seen in Table 8.

Table 8 – Excitation Schedule of Four Brick Masonry Column (Imai, 2011)

No.	Excitation	No.	Excitations
1	Step 0.05Hz ±1mm	7	JMA Kobe NS ±100mm
2	Step 0.05Hz ±1mm	8	JMA Kobe NS ±125mm
3	Step 0.05Hz ±1mm	9	JMA Kobe NS ±150mm
4	JMA Kobe NS ±25mm	10	JMA Kobe NS ±175mm
5	JMA Kobe NS ±50mm	11	JMA Kobe NS ±200mm
6	JMA Kobe NS ±75mm		

5.8.1.4. Experiment Results

Model A collapsed by excitation No.6 vibration by peeling at the top of the joint between the first layer 4 and 5 from the bottom layer. Model C collapsed by excitation No.10 vibration, significantly crack from the previous excitation. Model B collapsed by excitation No.11 vibration. The last collapsed was Model D, the wire mesh tearing, fell from the third layer from the bottom. In the author's view, in this test, the wire mesh acts as "*safety net*".

If the wire mesh is embedded in mortar, the strength of those columns will be increased because it is wrapped in ferrocement layer.

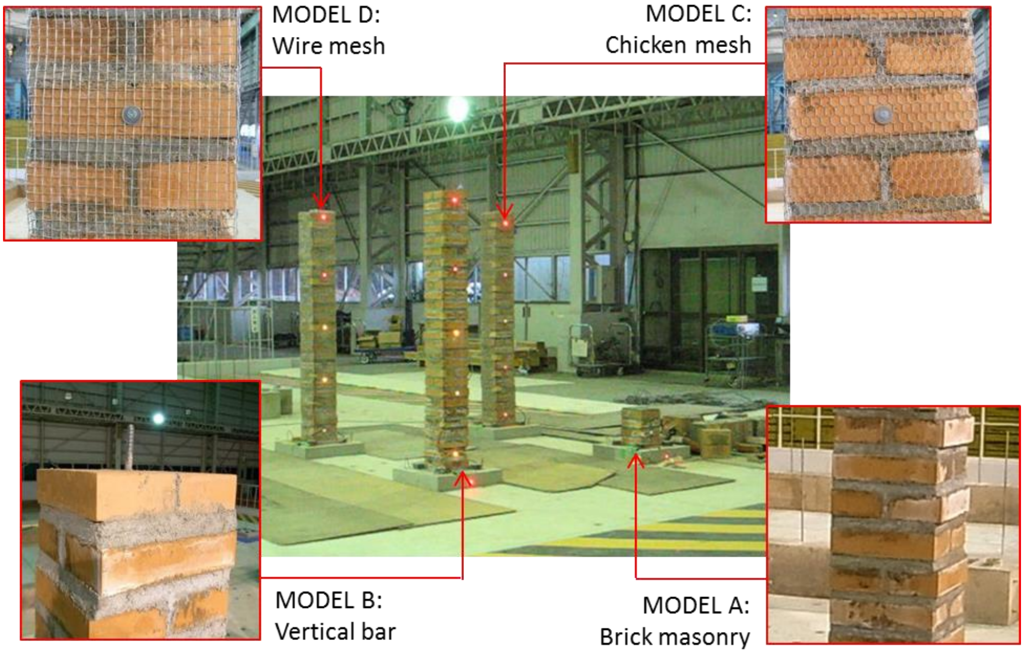
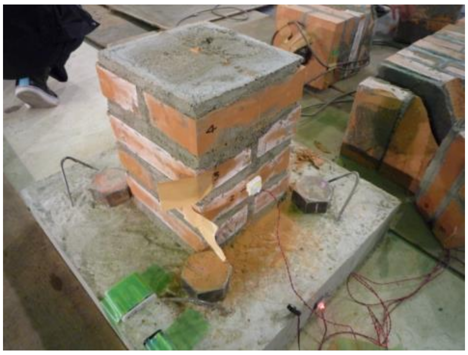


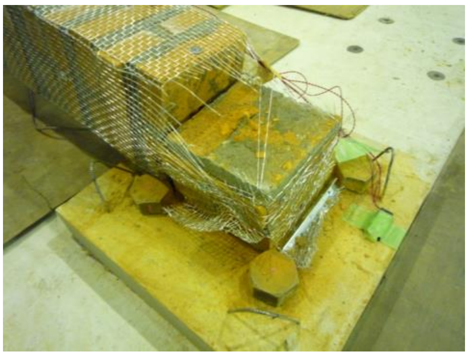
Figure 109 – Brick Masonry Column after Shaking by JMA Kobe 80% (Imai, 2011)



Model A

Model B

Model C



Model D

Figure 110 – State after the Collapsed of Four Specimens of Brick Masonry Column when Shaking by JMA Kobe 110% (Imai, 2011)

5.8.2. Shaking Table Test of Masonry Building Strengthened using Ferrocement Overlay based on Proposed Method (Boen, 2010)

In order to investigate the seismic behaviors of Indonesian brick masonry structures with or without reinforcement, the “Shaking Table Experiments on Full Scale Masonry Walls using Bricks imported from West Sumatra, Indonesia” was conducted as a collaborative research between National research Institute for Earth science and Disaster prevention (NIED) and Mie University (Imai & Nakatani, 2012; Hanazato, 2013). The bricks that were imported from Indonesia to Japan for the test were factory made.

In particular, the aim of this experiment is to assess the effects of the reinforcement using galvanized wire mesh on seismic performance and also to understand the seismic performance of both reinforced walls and unreinforced walls. The wire mesh used was the same as that was used to strengthen houses in West Sumatra and sent by the author from Indonesia.

The retrofitting method taken from guide books "Retrofitting Simple Buildings Damaged by Earthquakes”, published by UNCRD (Boen, 2010) and "Buku Panduan Perbaikan dan Perkuatan Bangunan Tembokan Sederhana" published by JICA (Boen, et al., 2012). The model was consulted first with the author and the test was performed after agreement is reached between the author and Mie University. Soon after the test, the results were relayed to the author.

All explanations below are excerpts from the report published by Mie University (Imai & Nakatani, 2012; Hanazato, 2013).

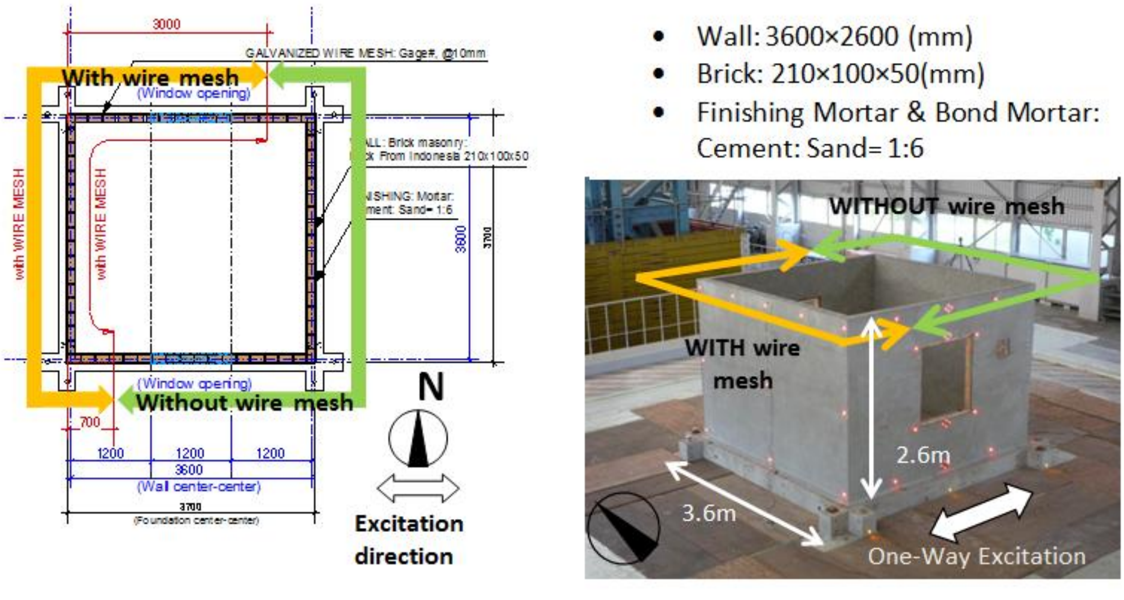
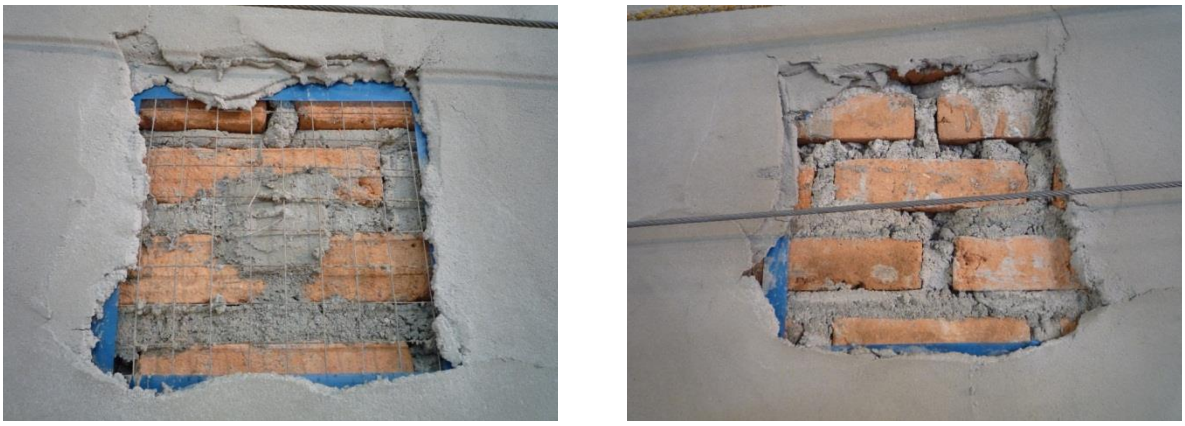


Figure 111 – Outline of Model Structure for Shaking Table Experiment of Reinforced Walls using Wire Mesh (Imai & Nakatani, 2012)

5.8.2.1. Structure Model

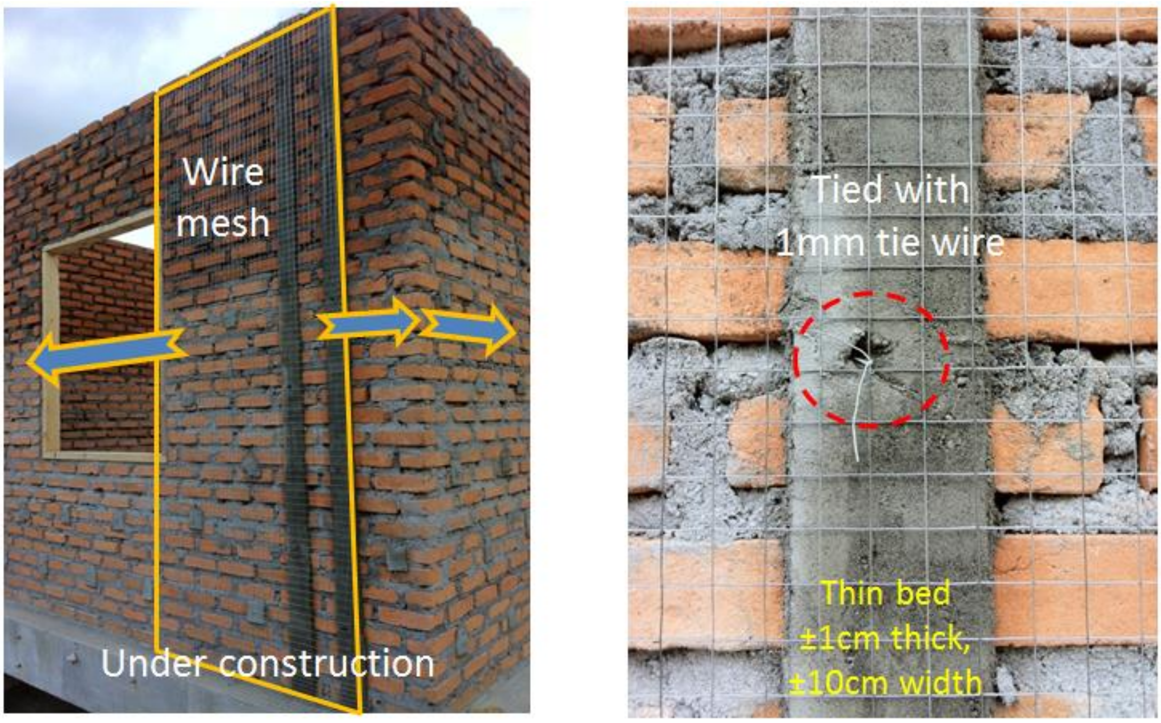
The model structure that was built in NIED, Tsukuba consisted of four walls with size 3600mm x 2600mm, made of bricks with mortar joints. The bricks were imported from

Padang, Indonesia, to reproduce the typical brick houses. The size of Padang's brick was defined L:210×W:100×T:50(mm) but the measurement of ten bricks showed that the average dimension was L:196.1×W:98.6×T:53.5(mm). The used wire mesh of 25mm grid also imported from Indonesia. For jacketing, both faces of inside and outside of brick walls were covered with mortar.



(a) *(b)*

Figure 112 – (a) Walls with Wire Mesh and (b) Walls without Wire Mesh (Imai & Nakatani, 2012)



(a) (b)

Figure 113 – Installing Wire Mesh based on Proposed Method (Boen, 2010; Imai & Nakatani, 2012)

5.8.2.2. Material Properties

The material properties tests of the specimen of mortar used to construct the model structure showed that compressive strength and tensile strength were 77kg/cm^2 and

4.5kg/cm^2, respectively. The compressive strength of brick was 39 kg/cm^2 with elasticity modulus 6,600kg/cm^2. The prism specimens composed of three pieces of bricks and joint mortar showed that the average compressive strength and shear strength were 19kg/cm^2 and 6.5kg/cm^2, respectively with average elasticity modulus 3,200kg/cm^2. Furthermore, the water absorption rate of brick was found to be 29.3 %. The adhesion tensile strength between brick and mortar was 3.7 kg/cm^2 (Imai & Nakatani, 2012).

5.8.2.3. Input Motions

The shaking table was a horizontal uniaxial movement type performed with 19 excitation cases as mentioned in Table 9.

Table 9 – Excitation Schedule of Full Scale Masonry Walls on June 25, 2012 (Imai & Nakatani, 2012)

Input No.	Input Wave	Accelerometer Measurement	3D Image Measurement	Crack Observation
1	STEP±1mm, 0.05Hz, 400gal	○	×	×
2	Sweep±1mm, 3-15Hz, 60s@5s	○	○	×
3	STEP±1mm, 0.05Hz	○	×	○
4	Sweep±1mm, 9-14Hz, continue	○	○	×
5	STEP±1mm, 0.05Hz	○	×	○
6	Sweep±1mm, 13-17Hz, continue	○	○	×
7	STEP±1mm, 0.05Hz	○	×	○
8	Sign±1.5mm, 10Hz	○	○	×
9	STEP±1mm, 0.05Hz	○	×	○
10	JMA Kobe NS ±87.5mm 50%	○	○	×
11	STEP±1mm, 0.05Hz	○	×	○
12	JMA Kobe NS ±175mm 100%	○	○	×
13	STEP±1mm, 0.05Hz	○	×	○
14	JMA Kobe NS ±200mm 110%	○	○	×
15	STEP±1mm, 0.05Hz	○	×	○
16	K-Net Ojiya EW 100%	○	○	×
17	STEP±1mm, 0.05Hz	○	×	○
18	JR Takatori 100%	○	○	×
19	STEP±1mm, 0.05Hz	○	×	○

Three dimension image processing was carried out to know the dynamic performance of the brick walls. 41 LED lamps and 4 high resolution cameras were used for measuring the dynamic response displacement of the model structure.

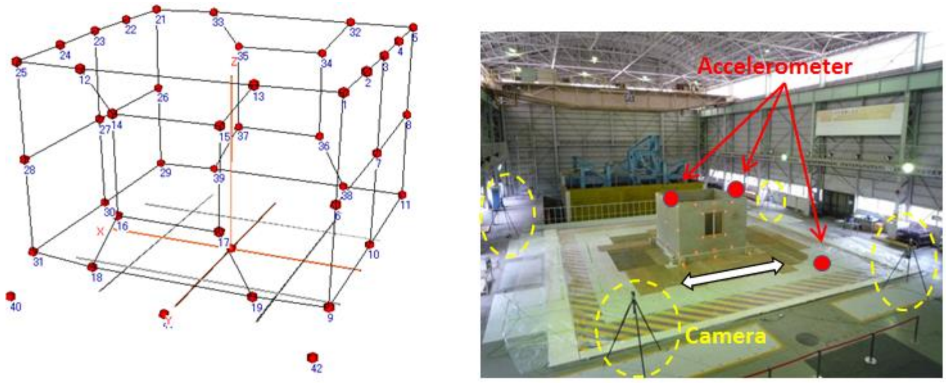


Figure 114 – Schematic of 41 LED Lamps Location and 4 High Resolution Cameras for 3D Image Measurement (Imai & Nakatani, 2012)

5.8.2.4. Experiment Results

The significant damage due to input motion can be seen in Table 10. Cracks were initiated from the corner of the opening and existing micro cracks. Out-of-plane without the opening did not have remarkable cracks. After shaken by input no. 12 (JMA Kobe 100%), the unreinforced wall collapsed.

Table 10 – List of Significant Damage of Strengthen Walls using Wire Mesh (Imai & Nakatani, 2012)

<table>
<tr><th>Input No.</th><th>Input motions</th><th>North</th><th>East (Unreinforced Wall)</th><th>South</th><th>West (Reinforced Wall)</th></tr>
<tr><td>2</td><td>Sweep 3-15 Hz</td><td>Minor cracks were happened</td><td>Cracks less than 10cm were happened</td><td>Minor cracks were happened</td><td>No damage</td></tr>
<tr><td>4</td><td>Sweep 9-14-9 Hz</td><td>A few small cracks were developed</td><td>A few small cracks were developed</td><td>Cracks were expanded</td><td>Vertical cracks were caused on the south side</td></tr>
<tr><td>6</td><td>Sweep 13-17-13 (Hz)</td><td>Remarkable cracks were caused from the corner of the opening on inside wall</td><td rowspan="3">Unchanged</td><td>Remarkable cracks were caused from the corner of the opening on inside wall</td><td>3 diagonal cracks 10cm were developed</td></tr>
<tr><td>8</td><td>Sign 10 (Hz)</td><td rowspan="2">Unchanged</td><td>Mortar start to crumble</td><td rowspan="3">Unchanged</td></tr>
<tr><td>10</td><td>JMA Kobe 50%</td><td>Wall around the opening bulged</td></tr>
<tr><td>12</td><td>JMA Kobe 100%</td><td>A part of wall collapsed along the existing vertical long cracks</td><td>All of wall collapsed</td><td>Half of wall collapsed along the opening</td></tr>
</table>

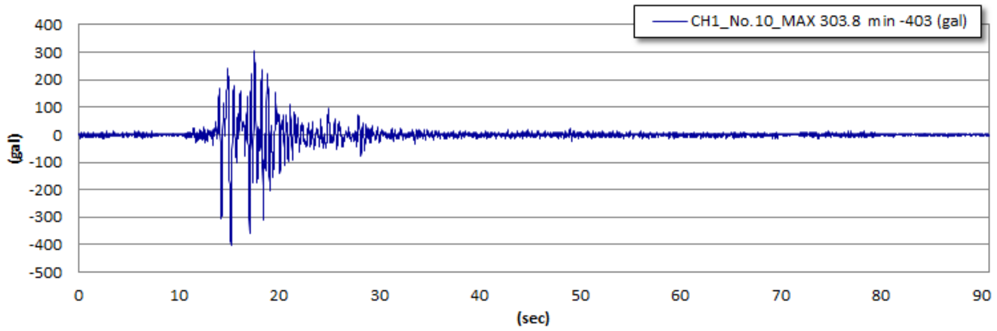


Figure 115 – Response Acceleration of Input No.10 JMAKobe NS 50% (Imai & Nakatani, 2012)

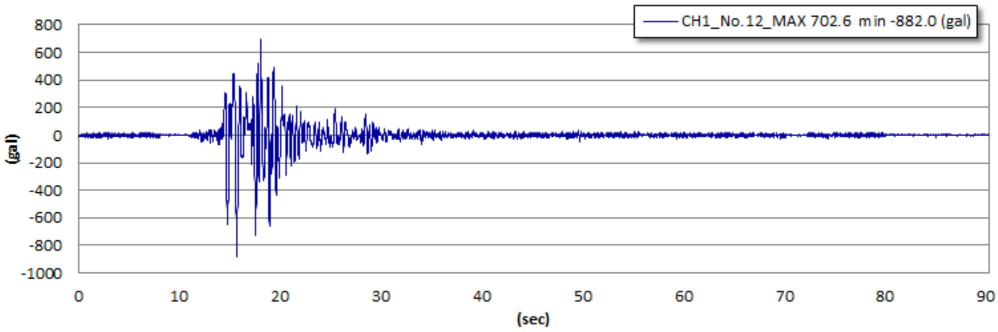


Figure 116 – Response Acceleration of CH1 Input No.12 JMAKobe NS 100% (Imai & Nakatani, 2012)

Figure 117 showed the observed cracks after shaken by Input No. 10 (JMA Kobe 50%) and Input No. 12 (JMA Kobe 100%). It was clear that the reinforced wall did not collapse, while unreinforced one collapsed after shaken by JMA Kobe 100%.

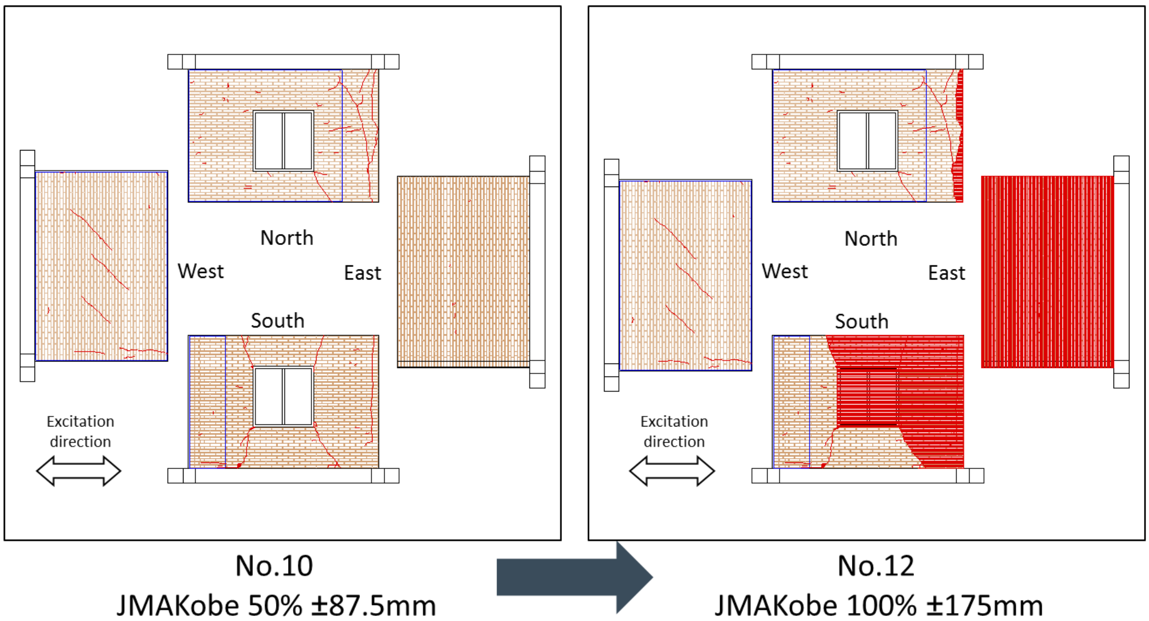


Figure 117 – Cracks on Walls after Shaken by Input No. 10 (JMA Kobe 50%) and Input No. 12 (JMA Kobe 100%) (Imai & Nakatani, 2012)

Figure 118 – Sequence Pictures of the Collapsed of Unreinforced Masonry Walls (Imai & Nakatani, 2012)

The maximum deformations in out-of-plane direction were 125mm and 55mm at the unreinforced wall and at the reinforced wall, respectively as can be seen in Figure 119.

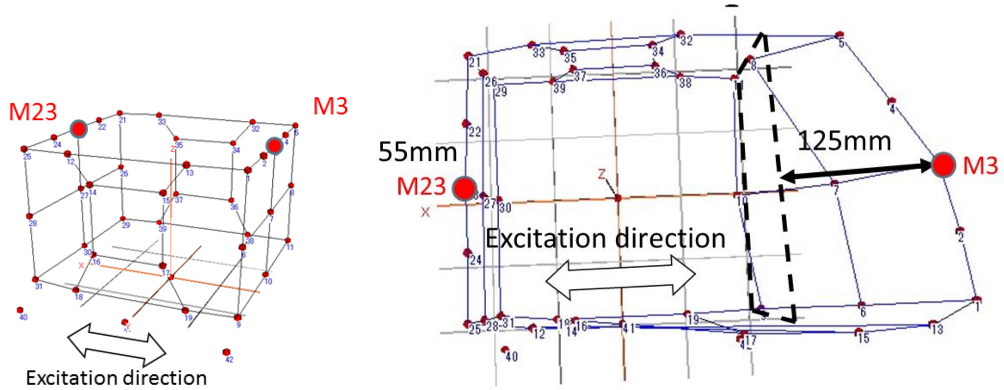


Figure 119 – Maximum Deflection in Out-of-Plane Direction after Shaken by Input No. 12 (JMA Kobe 100%) (Imai & Nakatani, 2012)

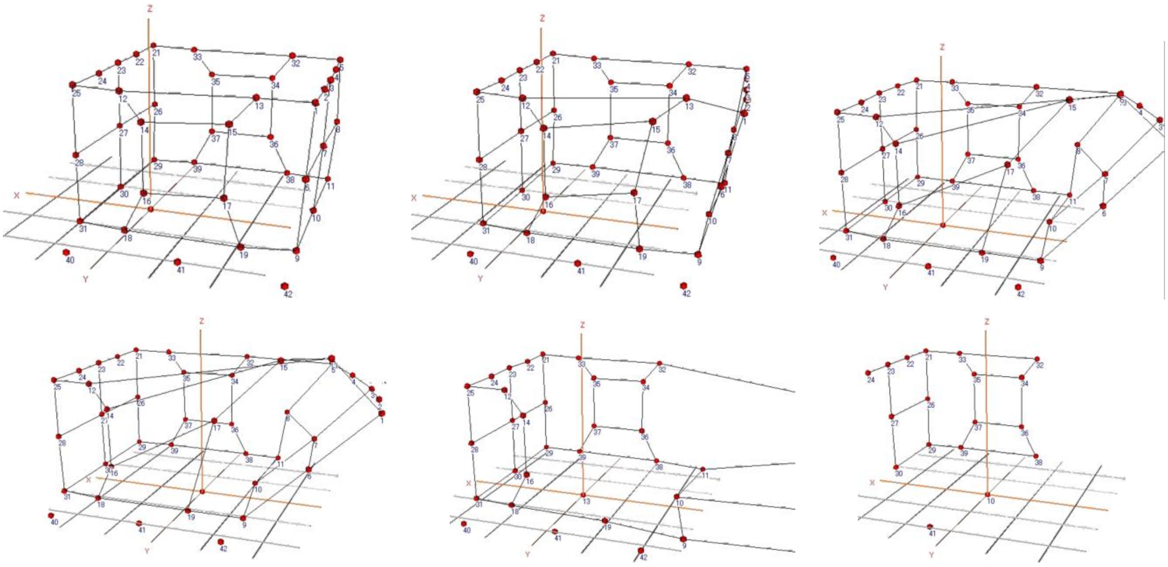


Figure 120 – Sequence of Collapsed of the Unreinforced Masonry Walls (JMA Kobe 100%) as Recorded by 41 LED lamps (Imai & Nakatani, 2012)

The significant effect of reinforcement using wire mesh was successfully demonstrated by the shaking table test. Predominant cracks in in-plane direction (the same direction in excitation) were initiated from the corner of the opening. Furthermore, the cracks occurred easily in unreinforced section of in-plane (North wall and South wall).

The shaking table test made it clear that the reinforcement using wire meshes was effective in preventing from collapse of wall.

5.9. Example of the Analysis and Design Utilizing an Existing Commercial Software

The analysis model or the analysis results may not be "exact" or "accurate" or "precise", meaning there are many uncertainties in modeling, analysis and design. The uncertainty may come from inaccurate material properties (mis-interpreting in bricks, mortar, masonry walls, concrete tests), inaccurate component dimensions (different bricks dimension, mortar thickness, etc.), or inaccurate or inappropriate strength formulas (lack of data to model inelastic behavior), particularly for masonry. However, this is not the most serious area of uncertainty. One of the most serious uncertainty is about the possible earthquake ground motion – its magnitude, frequency content, duration, probability of occurrence, and other things. The more information needed, the greater the uncertainty. Therefore, the analysis model or the analysis results must merely be good enough for making design decisions.

The method of analysis is using macro modeling principles, based on Strength Based Design (SBD) which assumed all elements are linear-elastic, homogeneous and isotropic material as explained in Chapter 5.5. The purpose of the analysis is not to simulate the actual behavior of the structure, which is impossible, but to obtain information that can be used in design. The goal is design or in other words to be able to make decisions.

5.9.1. Half-Brick-Thick Wall Panel Strengthened using Wire Mesh

A half-brick-thick wall panel 3.6m x 2.6m strengthened using wire mesh as shown in Figure 121(a) & (b) is analyzed using a commercial software SAP2000. The panel is fixed on 3 sides and free at the top side. The half-brick-thick wall panel consists of three main layers: ferrocement, brick-wall, and ferrocement.

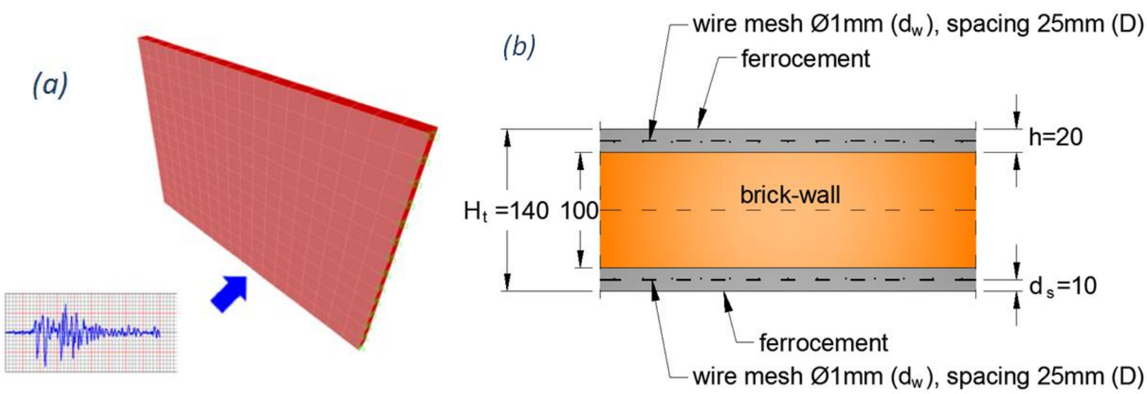


Figure 121 – (a) Half-Brick-Thick Wall Panel; (b) Section of Half-Brick-Thick Wall Panel

The analysis is linear-time-history. The 1995 JMA Kobe 100% is used as an input excitation. Material properties for this model are shown in Table 11.

Table 11 – Material Properties for Analysis of Brick-Wall Strengthened using Wire-Mesh

	Elasticity Modulus (kg/cm^2)	Compressive Stress (kg/cm^2)	Tensile Stress (kg/cm^2)	Shear Stress (kg/cm^2)
Brick-wall *)	3,343	5.9	0.5	6.5
Mortar **)	71,380	71.38	3.57	4.50
Ferrocement ***)	74,470	71.38	10.64	5.25

*) The elasticity modulus and shear stress of brick-wall is taken from one of the three elemental test that was conducted by National research Institute for Earth science and Disaster prevention (NIED) and Mie University (Imai & Nakatani, 2012). The compressive stress is derived from stress-strain relationship, $\sigma_c = E_c \times \varepsilon_c$. The tensile stress is taken approximately 5-10% from its compressive stress (Center for Building Technology, 1974).

**) The elasticity modulus and compressive stress of mortar are also taken from one of the three elemental test that conducted by National research Institute for Earth science and Disaster prevention (NIED) and Mie University (Imai & Nakatani, 2012). The tensile stress is assumed approximately 5% of its compressive (NBS Building Science Series 106, 1976, p. 147). Meanwhile the shear stress is calculated similar to reinforced concrete approach to estimate the contribution of mortar.

***) The material properties of ferrocement are calculated based on (ACI Committee 549, 1999; Naaman, 2000; Bangladesh National Building Code, 2012) which can be seen in Appendix E. With volume fraction of mesh in longitudinal direction 0.001571, the elasticity modulus of mortar 71,380kg/cm^2, and the elasticity modulus of wire mesh is 2,000,000kg/cm^2, the calculated elasticity modulus of ferrocement is 74,470kg/cm^2. The compressive stress of ferrocement is assumed equal to the compressive stress of mortar. With tensile stress of wire mesh 6,770 kg/cm^2 (based on test result that requested by the author in Bandung (see Chapter 5.7)), the tensile stress of ferrocement is 10.64kg/cm^2. The shear stress of ferrocement is calculated using semi-empirical approach (Desayi & Nandakumar, 1995), since (ACI Committee 549, 1997) only mention that ferrocement is used primarily in thin panels where the span-depth ratio in flexure is large enough that shear is not the governing failure criterion. Shear failure is preceded by the attainment of flexural capacity of ferrocement. The shear strength can be estimated approximately equal to 32% of its equivalent bending strength.

5.9.1.1. Bending Strength of Half-Brick-Thick Wall Panel

Using a simplified method to calculate ferrocement, the ultimate bending moment strength of the panel is 245.487(kg.m) (see Appendix D). Meanwhile, from SAP2000 analysis result, the maximum out-of-plane bending moment is 382.44(kg.m) located at the edge of the wall, as indicated in Figure 122 ( ).

Although the maximum out-of-plane bending moment that occurred is exceeded the calculated ultimate bending moment strength, it still can be said that the ferrocement panel itself (without taking account the strength of brick-wall) has significant bending strength to resist the earthquake load.

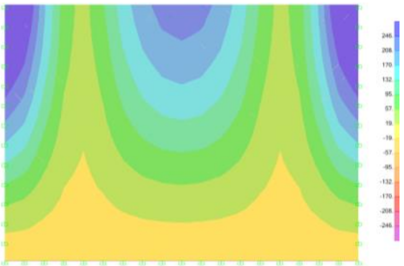
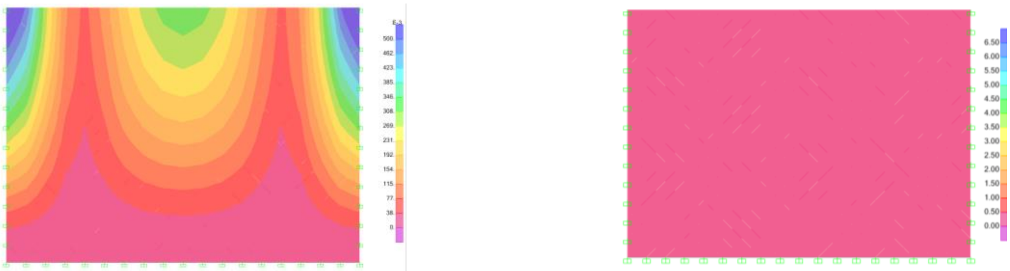


Figure 122 – Bending Moment Strength of Half-Brick-Thick Wall Panel

5.9.1.2. Stresses at Each Layer of Half-Brick-Thick Wall Panel

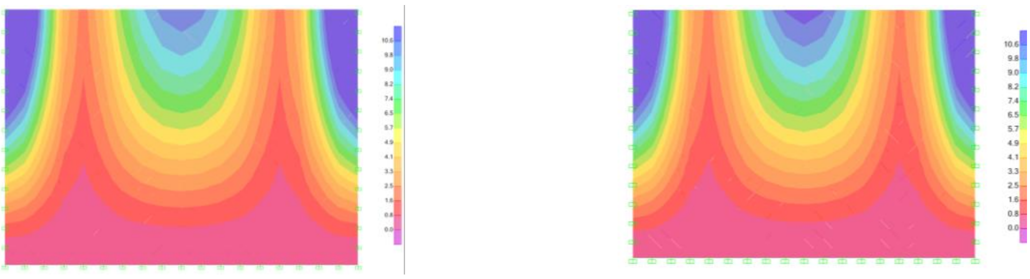
Figure 123(a) and Figure 124 show the results at maximum excitation. It can be seen that the tensile stresses at the edge of brick-wall (0.618kg/cm^2) slightly exceed the limit of tensile stress (0.5kg/cm^2). The tensile stresses at the edge of wall in ferrocement layer (19.345kg/cm^2) also exceeded the limit tensile stress (10.64kg/cm^2) ( ).

However, the maximum out-of-plane shear stresses, both in brick-wall and in ferrocement, are less than the limit of shear stress for each layer (Figure 123(a) and Figure 125).



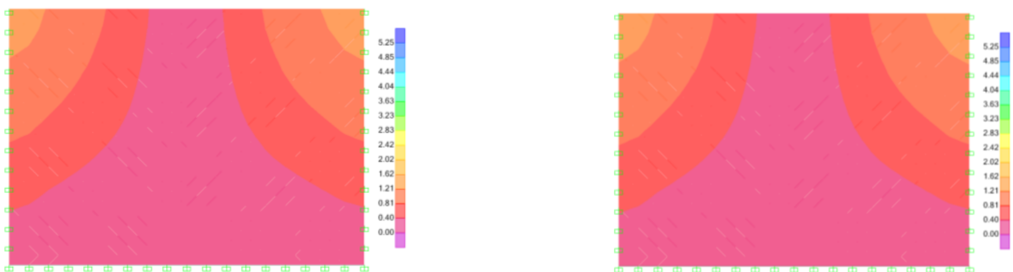
(a) *(b)*

Figure 123 – Stresses Pattern in Brick-Wall: (a) Tensile Stresses; (b) Out-of-Plane Shear Stresses



(a) *(b)*

Figure 124 – Tensile Stresses Pattern: (a) at Outer Ferrocement; (b) at Inner Ferrocement



(a) *(b)*

Figure 125 – Out-of-Plane Shear Stresses Pattern: (a) at Outer Ferrocement; (b) at Inner Ferrocement

5.9.2. *Simple Masonry Building Strengthened using Wire Mesh*

A computer analysis is performed for the masonry building strengthened using ferrocement overlay as described in Chapter 5.8.2, page 144 that was tested on the shaking table.

The analysis is linear-time-history. Material properties for this model are the same as used in previous example (see Table 11). Masonry brick-walls are modeled using layered-shell feature consist of 3 layers. For walls without wire mesh the layers are mortar, brick-wall, and mortar. For walls with wire mesh the layers are ferrocement, brick-wall, and ferrocement (composite of mortar and wire mesh).

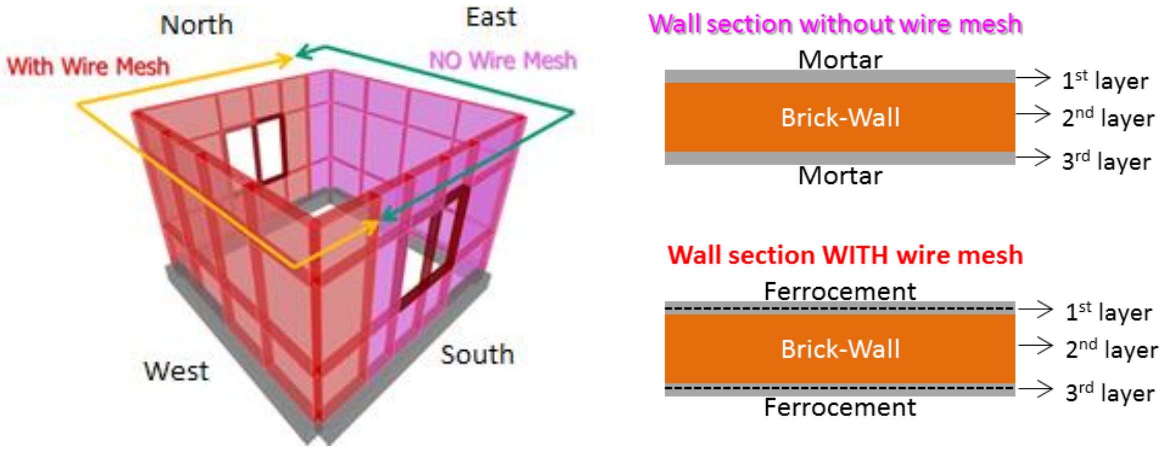


Figure 126 – (a) 3D Model of Masonry Building Strengthened using Wire Mesh

The 1995 JMA Kobe 100% is used as an input excitation as can be seen in Figure 127.

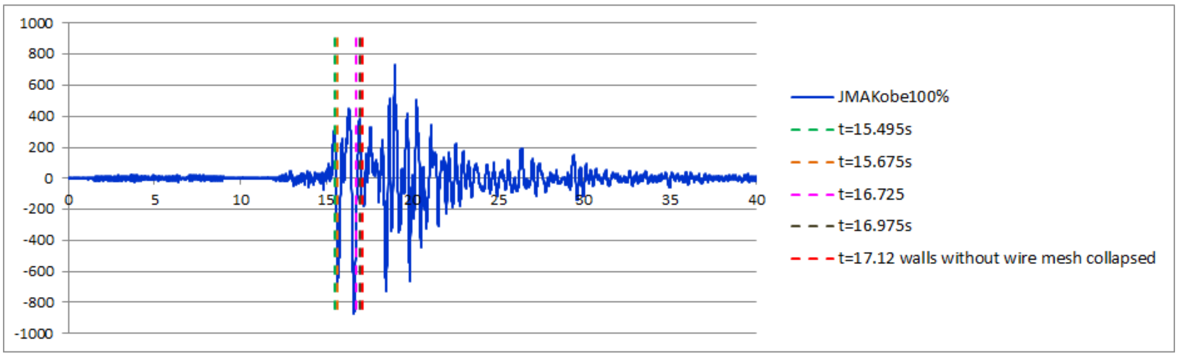


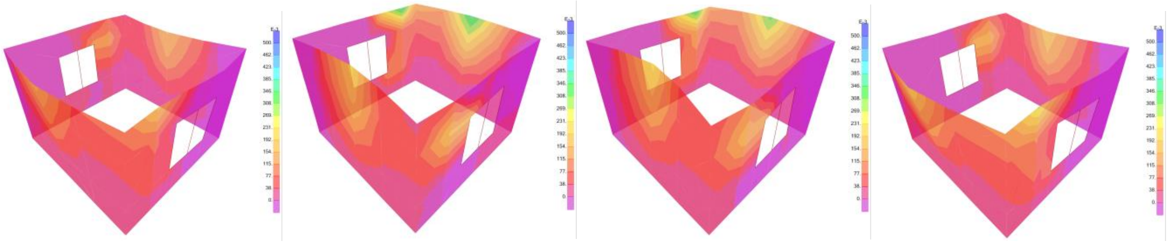
Figure 127 – Input Excitation for 3D Analysis of Simple Masonry Building Strengthened using Wire-Mesh

Figure 128 to Figure 131 show the sequential results of stresses in each layers of walls, before the walls without wire mesh collapsed at 17.12s.

indicates the stresses exceeded the permissible tensile stress, meaning substantial cracks start to develop in walls.

Figure 128 shows the tensile stresses in brick-wall panels. The stresses are smaller than the permissible tensile stress (0.5 kg/cm^2). Figure 129 shows the tensile stresses in inner and outer mortar layer (without wire mesh) in North wall. Figure 130 shows the tensile stresses in inner and outer mortar layer (without wire mesh) in South wall. Figure 131 shows the tensile stresses in inner and outer ferrocement layer (with wire mesh) in West wall.

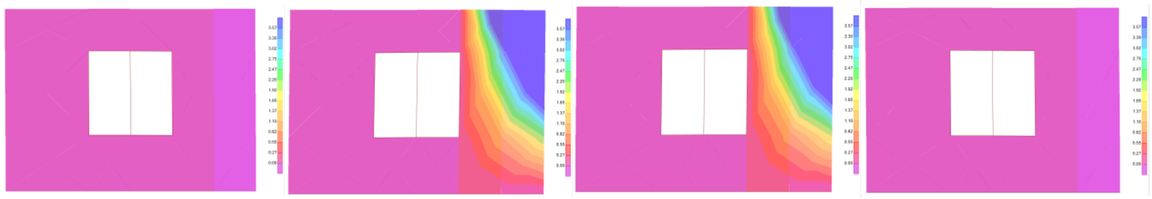
The out-of-plane shear stress of mortar layer and ferrocement layer are smaller than the limit shear stress (Figure 132 and Figure 133). Therefore, the out-of-plane shear stresses do not have significant contribution to cause the damage of wall.



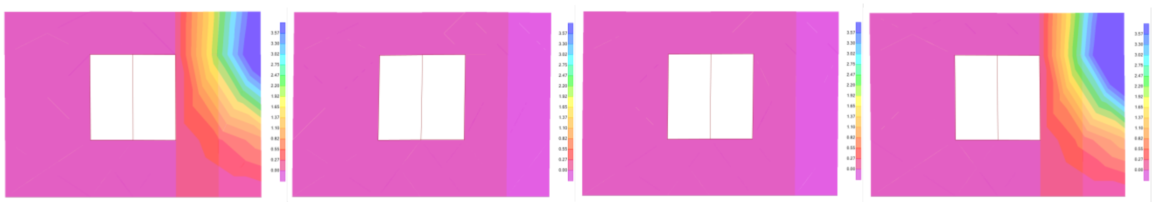
at 15.495s *at 15.675s* *at 16.725s* *at 16.975s*

Figure 128 – Tensile Stresses Pattern of Brick-Wall Layer

Inner Mortar Layer:



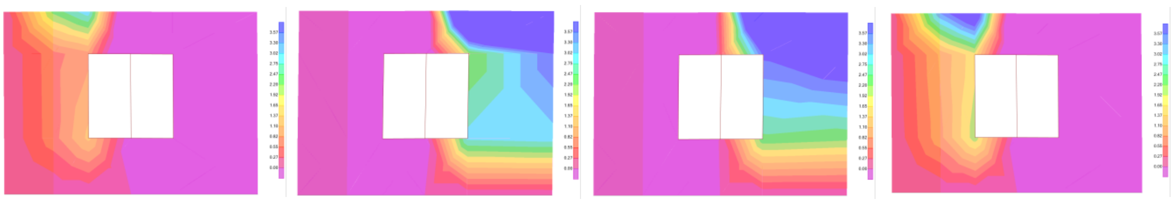
Outer Mortar Layer:



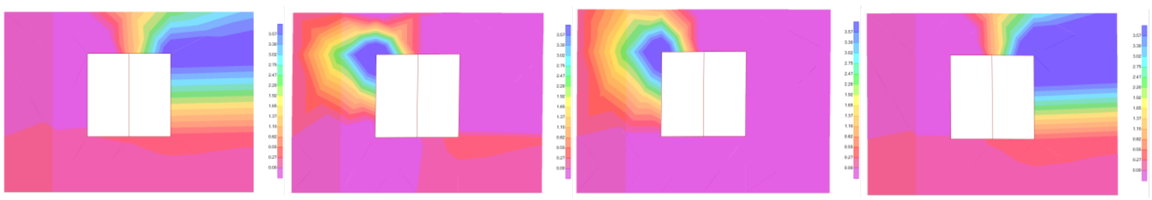
at 15.495s *at 15.675s* *at 16.725s* *at 16.975s*

Figure 129 – Tensile Stresses Pattern of Mortar Layer in North Wall (WITHOUT Wire Mesh)

Inner Mortar Layer:



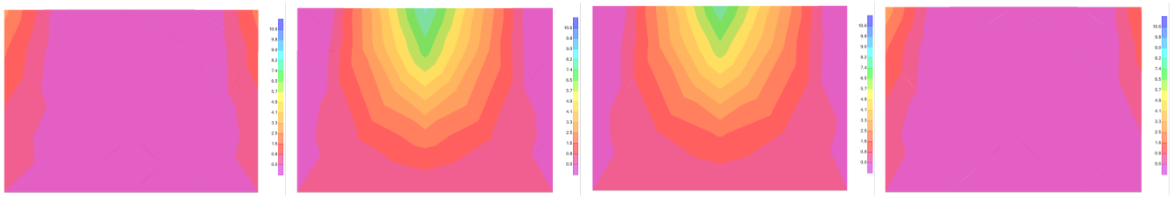
Outer Mortar Layer:



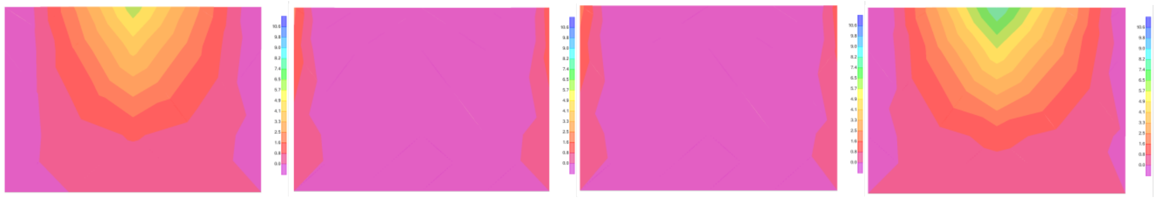
at 15.495s *at 15.675s* *at 16.725s* *at 16.975s*

Figure 130 – Tensile Stresses Pattern of Mortar Layer in South Wall (WITHOUT Wire Mesh)

Inner Ferrocement Layer:

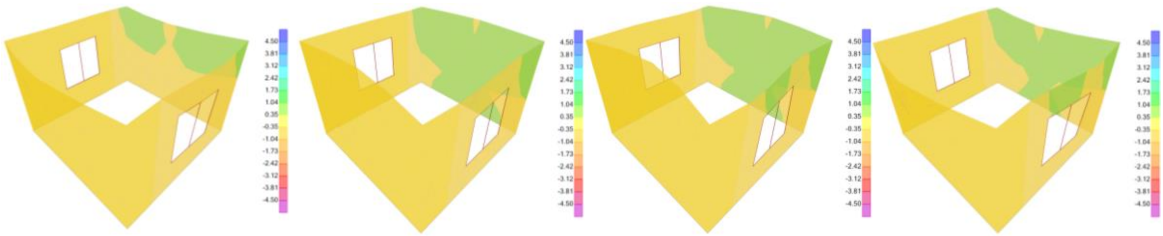


Outer Ferrocement Layer:



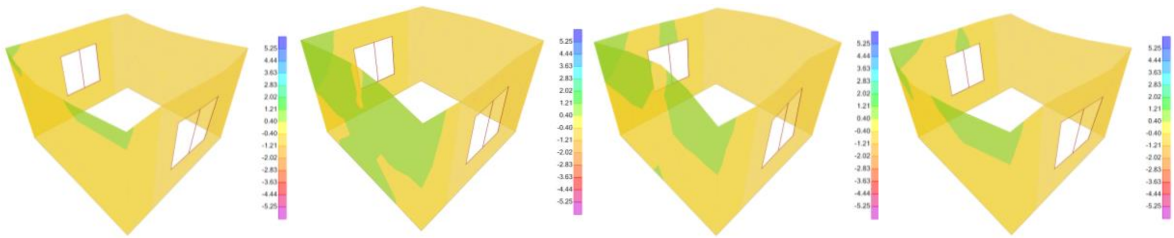
at 15.495s *at 15.675s* *at 16.725s* *at 16.975s*

Figure 131 – Tensile Stresses Pattern of Ferrocement Layer in West Wall (WITH Wire Mesh)



at 15.495s *at 15.675s* *at 16.725s* *at 16.975s*

Figure 132 – Out-of-Plane Shear Stresses Pattern at Mortar Layer (WITHOUT Wire Mesh)



at 15.495s *at 15.675s* *at 16.725s* *at 16.975s*

Figure 133 – Out-of-Plane Shear Stresses Pattern at Ferrocement Layer (WITH Wire Mesh)

5.10. Comparison between Analysis Results and Shaking Table Test

Since masonry is considered by many experts as a non-elastic, non-homogeneous and anisotropic composite structural material as mentioned in Chapter 5.4, page 132, it is not possible to determine the mechanical properties of existing masonry walls by testing their constituent materials in laboratory (Tomazevic, 1999). It is also difficult to reproduce the existing masonry walls in the laboratory, even though very thorough chemical and mechanical tests of the mortar, brick may have been carried out. Different variability, different tests, and different numbers of specimens will result in different test results. This is mainly due to the relatively brittle behavior of masonry and partly to a lack of sensitivity in the test and measurement techniques (NBS Building Science Series 106, 1976).

As mentioned earlier in Chapter 5.9, page 150, inaccurate material properties, such as elasticity modulus, compressive strength, strain, both for brick-wall as well as mortar and wire mesh, may cause uncertainties in modeling, analysis, and design. However, as mentioned in Chapter 5.4.3, page 134, for modeling load-bearing walls, masonry is assumed as isotropic since the result stress patterns of isotropic and anisotropic models are similar (Haadi & Dilrukshi, 2009). Therefore, it is difficult to duplicate the results from experimental with numerical analytical model using computers. **This is good enough until a more refined limit state verification method is established for masonry wall bearing structures that is easy to apply.**

There is a correlation between cracks that was observed from shaking table test as shown in Figure 117, page 148 with analysis results as shown in Figure 128 to Figure 131, page 154-155. From shaking table test, the cracks occurred at North wall and South wall which were not strengthened using wire mesh. With the increasing of load, the crack become larger and caused the wall collapsed. Cracks at North wall come from the boundary of North wall and East wall that made wall tend to tear apart; meanwhile from analysis result, the stresses of mortar layer in the boundary of North wall and East wall exceeded the permissible tensile stress (Figure 129), which indicates that cracks might have occurred. Furthermore, cracks at South wall come from the opening (Figure 118) which correlate with the analysis results (Figure 130) where the stresses of mortar layer also exceeded the permissible stress at the corner of opening. The West wall, which was strengthened using wire mesh, did not collapse; the stresses occurred at ferrocement layer (Figure 131) also do not exceed the permissible stress.

As explained in Chapter 4.5, it must be reiterated that the purpose of the analysis is not to simulate the actual behavior, but to get reliable information that there is a correlation between the observed damages and the results of the analysis. The correlation is not perfect, but is good enough to get a good idea to build appropriate non-engineered constructions that can withstand earthquakes.

Tests results for mortar and brick-wall compressive strength and elasticity modulus conducted in Japan gives very different results than the same test done in Indonesia as can be seen in Table 12.

According to (United Nations Industrial Development Organization (UNIDO), 1978), elasticity modulus of mortar is 1,000 times its compressive strength and elasticity modulus of brick is 300 times its compressive strength. According to (Tomazevic, 1999), elasticity modulus of brick-wall masonry can vary from 200 to 2,000 times compressive strength.

Due to the numerous possible combinations of masonry bricks or units with masonry mortars, the range of obtainable wall strengths is very broad; say 10 to 500 kg/cm^2 (Sahlin, 1971). The brick masonry strength normally is about 25-50% of the brick strength; 25% referring for low strength mortar, and 50% referring for high strength mortar.

Therefore, results of analysis must be interpreted carefully and judgment is necessary.

The properties of the masonry wall, the mortar etc. cannot be obtained from the shaking table test, while those properties obtained from laboratory tests might not be the same as those in the actual masonry walls that are being tested on the shaking table.

It was stated earlier that a relatively elementary analytical model will suffice to predict failure. In more complex situations involving non-homogenous stress fields with large stress gradients and complex deformation fields, a more detailed analysis may be necessary. It is for this purpose that the micro-modeling is being pursued. Finite element simulations of panel behavior have been performed to assess the accuracy of current micro-modeling concepts. For this purpose, the panel assembly is discretized into a system of plane stress finite elements. However, from a practical standpoint, it is imperative that one be able to predict basic macro-element properties from component properties. Extensive testing has indicated that this is very difficult. Several examples are provided below with respect to the failure surface described previously.

We must be realistic in laboratory and job tests and material requirements. Mortar tests as per ASTM are fine for testing the cement, but they do not provide information as to the performance or influence of mortar in a structural wall. Prisms are an indicator of compressive strength, but they must be representative of the actual masonry wall strengths.

Table 12 – Compressive Stress & Elasticity Modulus of Mortar, Brick, and Brick-Wall

	Mortar				Brick			Brick-wall			
	Mortar Mix	Comp. Strength, f_j (kg/cm^2)	Modulus Elasticity, E_j (kg/cm^2)		Comp. Strength, f_b (kg/cm^2)	Modulus Elasticity, E_b (kg/cm^2)		Comp. Strength, f_m (kg/cm^2)	Modulus Elasticity, E_m (kg/cm^2)		Shear Strength, f_v (kg/cm^2)
		Test	Test	E_j = 1,000 f_j	Test	Test	E_b = 300 f_b	Test	Test	E_m = 200~2,000 f_m	
Sven Sahlin, 1971								25%-50% f_b		700 f_m	2%-15% f_m
Unido, 1978	1pc:1lm:6ps	46.58	110,000	46,580	27.85	10,788	8,355	43.8	-	8,760 ~ 87,600	1.7
IAEE, 1986	1pc:3ps 1pc:6ps 1pc:12ps				70	-	21,000	42 33 24	10,000	6,475 ~ 64,750	10.4 3.9 2.2
Tomazevic, 1999	-	50	-	50,000	25	-	7,500	15	38,000	3,000 ~ 30,000	12
Arya, 2007 (Indian bricks)	1pc:3ps 1pc:6ps	-	-	-	-	-	-	94.4 61.3	-	18,880 ~ 188,800 12,260 ~ 122,600	10.7 3.87
Minowa, 2010 (test Dec 27, 2007) (Pakistan brick)	1pc:8ps	92	11,000	92,000	147	77,000	44,100	-	-	-	-
Minowa, 2010 (test July 4, 2008) (Japanese brick)	1pc:8ps	25.8	25,000	25,800	298	83,000	89,400	-	-	-	-
Puskim-JICA, 2010	1pc:4ps	197.8	7,820	197,800	38.4	644.1	11,520	-	-	-	2.1 ~ 6.2
Svetlana, 2011		40 ~ 125		40,000 ~ 125,000	40	-	12,000	15	-	3,000 ~ 30,000	3.0 ~ 3.5
Puskim-JICA, March 2012	1pc:4ps 1pc:7ps	89.4 75.1	-	89,400 75,100	43.3 ~ 58	-	12,990~ 17,400	-	-	-	-
Imai, 2012	1pc:6ps	77	70,000	77,000	39	6,581.6	11,700	19	3,343	3,800 ~ 38,000	6.5

pc = cement ; ps = sand ; lm = lime

Chapter 6 Discussion and the Way Forward

Non-engineered masonry constructions in Indonesia and its problems due to earthquakes have been elaborated. The damaged and/or collapsed of this type of buildings caused many casualties and economic loss at every earthquake because the buildings are not earthquake resistant. Therefore, this problem must be resolved as quick as possible to prevent further fatalities and loss in the next disaster.

6.1. Retrofitting

As mentioned in Chapter 1.1, all of the damages due to earthquake disasters to date are repetitions of all past occurrences and are a demonstration that in Indonesia not much has been done with regard to non-engineered constructions. Figure 15 showed that there are millions of non-engineered constructions not earthquake resistant.

Many of the damaged non-engineered constructions do not have to be demolished but can be retrofitted. From an economic point of view, it is unreasonable to rebuild all the structures that cannot withstand earthquakes, although such an action is ideal. Chapter 1.3.2.4 clearly explained the benefits of retrofitting.

Chapter 5.3 explained the proposed retrofitting method using ferrocement layers as the skin of a sandwich structure. It is a **simple, affordable, replicable** method, and suiting the local culture as demonstrated during retrofitting works of several schools in West Java (2006 and 2007), Padang (2010), and the latest two houses in West Sumatra, namely in Tanjung Basung (2012) and in Gadur (2013). In Tanjung Basung, the ferrocement layers were applied for the entire surface of the walls. In Gadur, the ferrocement layers were only applied as substitute of “practical” columns and “practical” beams 50cm width and diagonally.

6.1.1. Retrofitting a House in Tanjung Basung

In December 2012, a non-engineered masonry house was retrofitted based on the proposed retrofitting method. The retrofitting work was sponsored by JICA. Figure 134 shows the layout of the retrofitted house in Tanjung Basung, Pariaman, West Sumatra. The total footprint of the house is 59.185m^2. The ferrocement layers were applied on both sides for all the walls.

Figure 135 shows the sequence of retrofitting works in Tanjung Basung. The detail explanation of this proposed retrofitting method can be seen in Chapter 5.3 on page 122.

The pictures give an idea of the simplicity, applicability, and replicability of the proposed retrofitting method. The local masons with average skills were trained in situ for approximately 30 minutes and subsequently can do the job by themselves. The training was only for how to make the support for the wire mesh, using plaster or umbrella-head-roofing-nails and in fastening the wire mesh to those supports. The local masons are familiar with plastering works.

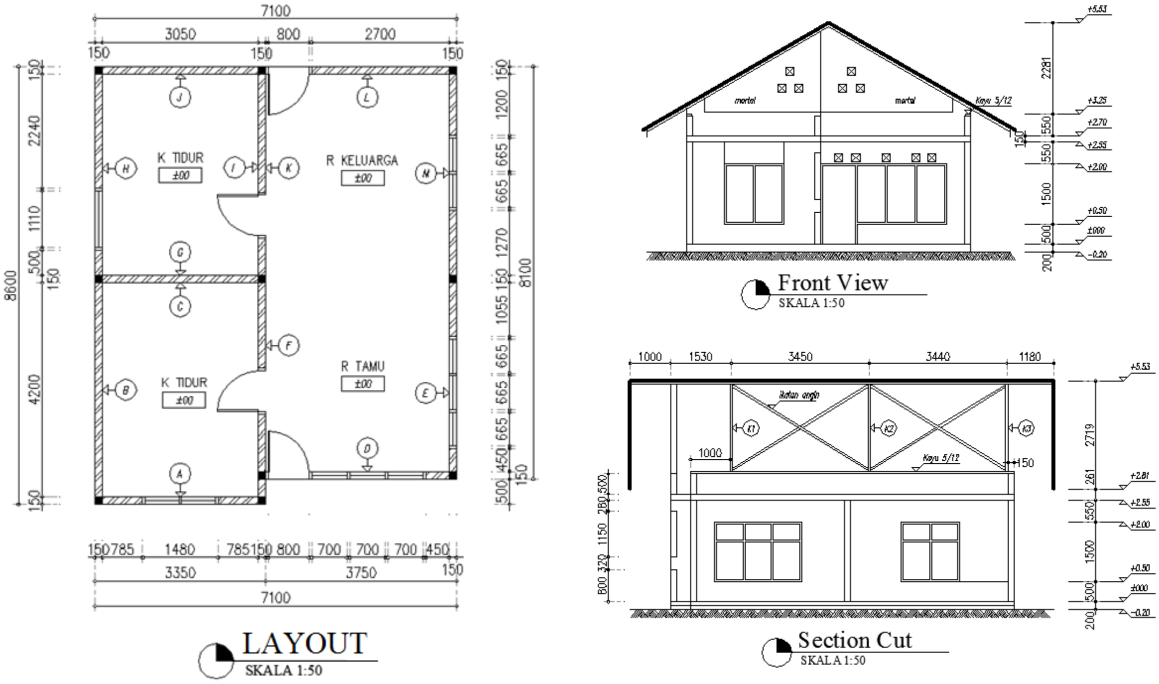
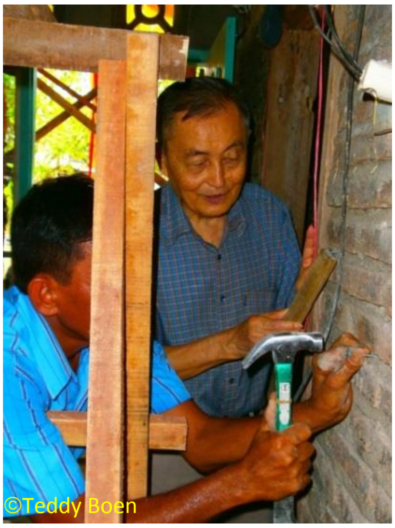


Figure 134 – Layout of Retrofitted House in Tanjung Basung



Installation Umbrella-Head-Roofing-Nail as Supports of Wire Mesh

Installation of Wire Mesh, Tie to Umbrella-Head-Roofing-Nail

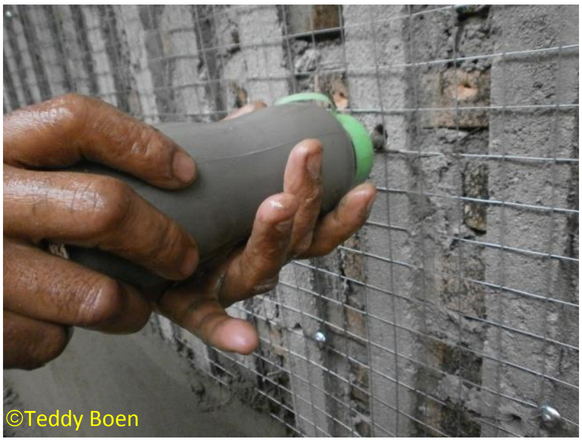
Wire Mesh Installed and Fastened to Top of Umbrella-Head-Roofing-Nail

Figure 135 – Sequence of Retrofitting Works in Tanjung Basung

Drilling the Walls for Stitching

Stitching the Inside and Outside Wire Mesh

Trained Local Masons Stitching the Inside and Outside Wire Mesh



Grouting of Drill Holes using Cement Paste

Re-plastering the Wall

Figure 135 (cont'd) – Sequence of Retrofitting Works in Tanjung Basung

Figure 136 – Before and After Retrofitting, House in Tanjung Basung

Table 13 shows the approximate retrofitting cost of the house in Tanjung Basung if done properly, i.e. Rp 354,000.00 per m^2.

The market price at that time (December 2012) to build a new simple houses in Padang without land-price is approximately Rp 1,500,000.- per m^2. The cost to build a new house with 59.185m^2 footprint similar to the house in Tanjung Basung is approximately Rp 89,000,000.-

Therefore, the retrofitting cost is approximately 23.6% if compare with the cost to build the new house. The average retrofitting cost should be in range of ±15-20%. In this particular case, the wall height is above normal (total wall area is 140.23 m^2).

Table 13 – Approximate Retrofitting Cost in Tanjung Basung if Done Properly – December 2012

No	Description	Unit	Volume	Unit Price (Rp)	Total Cost (Rp)
1	Cement water mix	Ls	1.00	Rp 200,000	Rp 200,000
2	Installation of wire mesh	m^2	280.46	Rp 28,066	Rp 7,871,501
3	Re-plaster brick-walls	m^2	280.46	Rp 45,966	Rp 12,891,689
				TOTAL COST	**Rp 20,963,190**

6.1.2. Retrofitting a House in Gadur

In January 2013, once again JICA sponsored the retrofitting of a non-engineered masonry house in Gadur, Pariaman, West Sumatra using the proposed retrofitting method. Figure 137 shows the layout of the retrofitted house. The total footprint of the house is 61.6m^2. The ferrocement layers were only applied as substitute of “practical” columns and “practical” beams 50cm width and diagonally. Figure 138 show the sequence of retrofitting works in Gadur. Local masons were also trained similar to that in Tanjung Basung, namely on how to install umbrella-head-roofing-nails and fastening the wire mesh. Similar to Tanjung Basung, the local masons can absorb what is being taught within several minutes only.

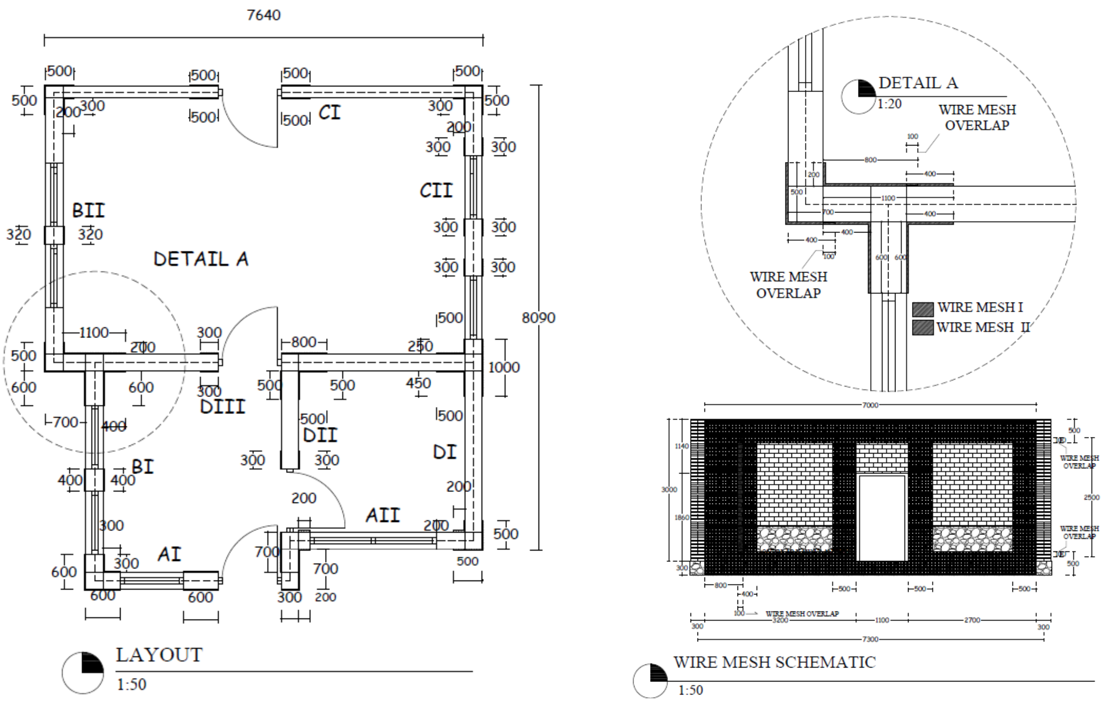


Figure 137 – Layout of Retrofitted House in Gadur

Sealing the Cracks with Cement - Sand Mortar

Spraying Walls with Cement-Water

Installation Umbrella-Head-Roofing-Nail as Supports of Wire Mesh

Figure 138 – Sequence of Retrofitting Works in Gadur

Installation of Wire Mesh, Tie to Umbrella-Head-Roofing-Nail

Wire Mesh Installed and Fastened to Top of Umbrella-Head-Roofing-Nail

Diagonal Ferrocement Layer

Ferrocement as Foundation Beams

Drilling the Walls for Stitching

Stitching the Inside and Outside Wire Mesh

Figure 138 (cont'd) – Sequence of Retrofitting Works in Gadur

Grouting of Drill Holes using Cement Paste

Re-plastering of Wall

Figure 138 (cont'd) – Sequence of Retrofitting Works in Gadur

Figure 139 – Before and After Retrofitting, House in Gadur

Table 14 shows the approximate retrofitting cost of the house in Gadur if done properly, i.e. Rp 236,000.00 per m^2.

The market price at that time (January 2013) to build a new simple houses in Padang without land-price is approximately Rp 1,750,000.- per m^2. The cost to build a new house with $61.6m^2$ footprint similar to the house in Gadur is approximately Rp 107,800,000.-

Therefore, the retrofitting cost is approximately 13.5% if compared with the cost to build the new house. As explained earlier, the average retrofitting cost should be aprroximately ±15-20%. The footprint of the Tanjung Basung house ($59.185m^2$) is approximately the same as the houses in Gadur ($61.6m^2$). However, the total wall area in Tanjung Basung is $140.23m^2$ compare to the total wall area in Gadur which is $112.085m^2$.

Table 14 – Approximate Retrofitting Cost in Gadur if Done Properly – January 2013

No	Description	Unit	Volume	Unit Price (Rp)	Total Cost (Rp)
1	Patching of cracks	Ls	1.00	Rp 200,000	Rp 200,000
2	Remove plaster layer on both side of walls	Ls	1.00	Rp 200,000	Rp 200,000
3	Cement water mix	Ls	1.00	Rp 200,000	Rp 200,000
4	Installation of wire mesh	m^2	136.61	Rp 28,066	Rp 3,834,071
5	Re-plaster brick-walls	m^2	224.17	Rp 45,966	Rp 10,304,434
				TOTAL COST	**Rp 14,538,505**

In the author's opinion, those two actual retrofitting projects clearly indicates the retrofitting method propose is simple, applicable, replicable, and reasonable cost. The cost involved in similar retrofitting in West Java (2006 and 2007), Padang (2010) are all in the similar range of cost and also applied by local masons.

6.2. Applicability of the Proposed Retrofitting Method in Other Countries

Since the retrofitting method is simple, affordable, replicable and can be implemented by self-help construction, it can also be applied in other earthquake prone developing countries such as Bangladesh, India, Myanmar, Nepal, Pakistan that have some similarities in non-engineered masonry construction as that of Indonesia, which is shown in Figure 140 (Center for Disaster Mitigation Institute Technology Bandung, 2011):

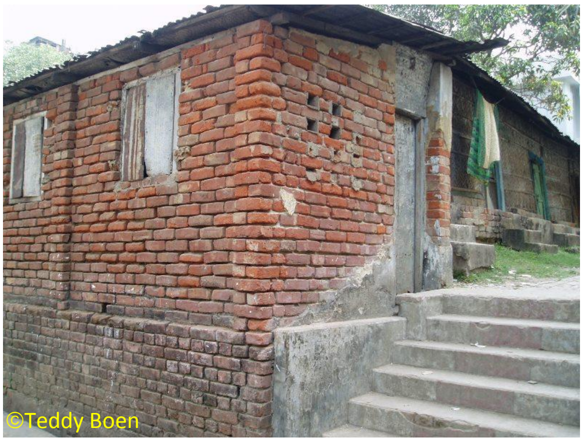
1. Design intervention by owner
2. Building owner are private
3. Almost all types of materials and workers are available
4. No supervision by other parties, supervised by owner themselves
5. Using small concrete mixer or mixed by hand for concreting
6. The buildings function as residential
7. Use strip foundation
8. Half-brick or one-brick wall thickness
9. Use Portland cement for wall plaster
10. Use Portland cement as cement material for concreting
11. No training for the workers

Most of the non-engineered constructions in developing countries, technically, are not properly constructed. Many building owners and craftsmen have limited knowledge on proper construction methods and they do not consider earthquake as a potential hazard. Most of the owners put deeper attention to the construction cost rather than building safely. Some of the craftsmen / masons have relatively insufficient formal education or training on proper building construction and gained their skills only from both the guidance from the foreman and their own experiences (Okazaki, et al., 2012).

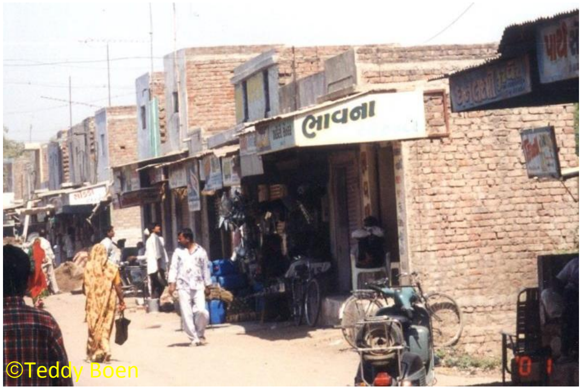
From the survey results, funded by National Graduate Institute for Policy Studies – GRIPS, most of the non-engineered constructions do not pay attention on the detailing, quality of materials, and quality of workmanship. Those findings are similar to Indonesia as described in JICA report explained in Chapter 3.6.1.

Therefore to reduce the earthquake risk in the future, all of those non-engineered construction should be reviewed and retrofitted if necessary. Since the non-engineered construction in developing countries has similarities as mentioned above, the retrofitting techniques introduced in this dissertation are valid and can be applied.

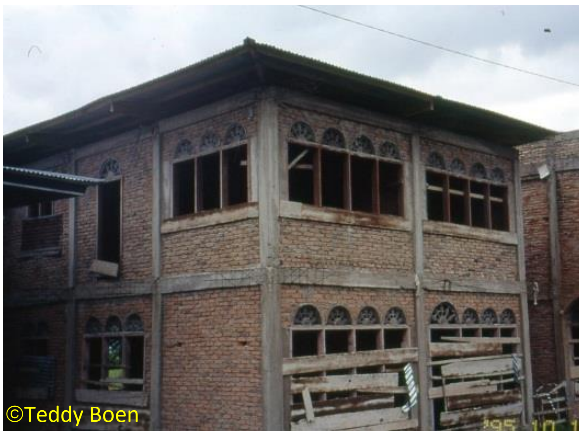
The proposed retrofitting technique can also be applied in engineered buildings to strengthen the walls.



Afghanistan *Bangladesh*

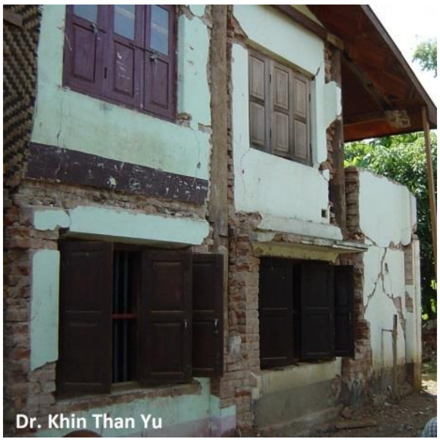


India

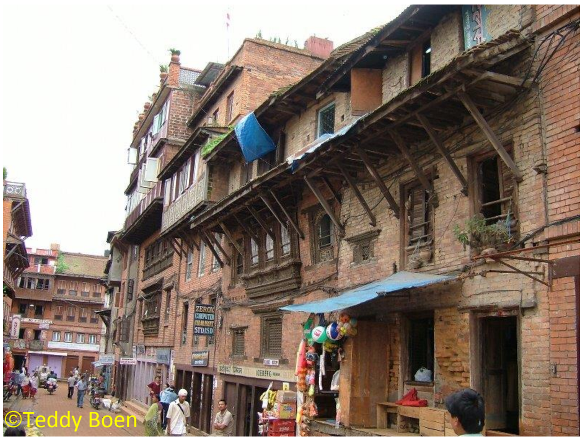


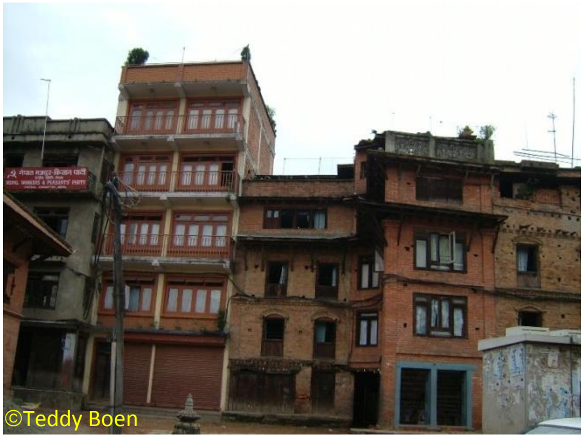
Indonesia

Figure 140 – Non-Engineered Construction in Developing Countries



Myanmar





Nepal

Pakistan

Figure 140 (cont'd) – Non-Engineered Construction in Developing Countries

6.3. Needed Improvements

- ***Law Enforcement***

Laws are restrictions for the government as well as the communities, however, law is important in setting safety standards and law is a vital element in public education. The problem in Indonesia is LAW ENFORCEMENT that is not working properly, and so far Indonesia can not afford to set up an education system to enforce law consistently (Chapter 2.2.4). Therefore, this situation resulted in legal control loses their essential public respect.

Unless law enforcement is improved, little can be expected that disaster risk reduction measures can be successfully applied.

- ***Political Will and Leadership***

To introduce, develop and maintain disaster risk reduction measures and all related activities, a strong pressure from centers of political power in needed. Generally, political will originate after a big disaster, after a major failure occurred. However, if the public are informed about the nature of the hazards in their respective areas, the vulnerability and what safety measures can be adopted to safe their life and property, such public awareness serves as a continuous pressure on the government to come up with solutions for protective actions. This also can result in the government's political will. So far in Indonesia, this does not occur (Chapter 2.1.1 and 2.2.6) and therefore, Indonesia needs a strong management, leadership and skills in all levels of the government as well as private sectors to establish an effective implementation of disaster risk reduction. In order for the government to possess political will, it is necessary to stimulate awareness among national and local planners in disaster stricken areas in Indonesia so that they in turn include disaster risk reduction measures in their respective areas;

- ***Disaster Risk Reduction***

Chapter 2.2 clearly indicates that disaster risk reduction in Indonesia is not yet optimal and must be drastically improved. Disaster risk reduction must be created by hard work, knowledge within the governmental and non-governmental organizations, therefore, BNPB, BPBD, government ministries and the government administrations must work hard to create an effective disaster risk reduction policy. It is strongly suggested that BNPB, BPBD, and all related government ministries respond as systematic as possible to disasters to prevent wasting of resources. Disaster risk reduction plans shall be based on risk assessment to indicate the scale of plan response that is necessary, planning that leads to the adoption and implementation of measures and strategies to reduce the risks.

However, all the actions must be spear headed by the government administration. If the government administration is functioning properly, it will affect the efficiency and nature of all disaster risk reduction related activities.

In Indonesia, the risks assessment has not been done accurately and in such case, most rely on intuitive assessment. However, even without a systematic risk assessment, a systematic and planned response must still be followed. On the other side, over-planning is not advised, over preparedness is costly and disaster risk reduction measures will be discredited if the disaster does not occur.

Although a systematic risk assessment and a systematic planning are encouraged, planning and decision making must be based on judgments that require skills in many disciplines since planning for disasters is not an "exact" science that can follow textbooks. For this reason, it is high time that all government organizations employ professionals with expertise and track records in disaster related problems.

Note: In many literatures, it is mentioned that if earthquake disaster risk reduction measures are to have any success, they must be integrated into the ongoing development process. All related programs should have an integral focus taking into account not only earthquake disaster risk reduction but also education, food, employment, housing, and other basic needs and must be in accord with the social, cultural and economic context. Attitudes to risk assessment vary from the ideal to the pragmatic. The ideal approach is as stated above, which claims that education, food, employment, housing, and other basic needs are the underlying cause of vulnerability and that therefore, the precondition for disaster risk reduction measures to succeed is the fulfillment of the above mentioned shortcomings. However, such preconditions are impractical. Communities, urban as well as rural require protection against disaster irrespective of the above mentioned shortcomings. It is a fact that the socially disadvantage are the most vulnerable to the disasters due to precarious conditions. They live on poor quality land and cannot build safely. Therefore, the practical action to take is to help administrations and the people to improve building practices and siting, and introduce basic technical precautions to reduce identifiable risks, meaning to lower the risk by improving building practices or by moving to other sites, or both up to an acceptable level of risk.

- ***Continuity in Implementation***

In Indonesia, one of the drawback of implementing strategies, including but not limited to risk reduction strategies, is the discontinuity when the lifetime of political administrations term ends. The new administration almost always has new policies and new priorities of resource allocation ideas. Even good programs instigated by the previous administration are downgraded or even terminated even though politically acceptable. To ensure long term objectives, all reforms must be institutionalized and make BNPB and BPBD as an independent agency with the responsibility to promote disaster risk reduction, including seismic safety.

- ***Roles and Responsibilities***

Disaster risk reduction is very wide in scope and complex relationship within government ministries and agencies (Chapter 2.2.1.1). To get an effective disaster preparedness planning and risk reduction, there must be a clear allocation of roles and responsibilities. Therefore a clear definition of tasks, roles and responsibilities must be defined in great detail and provide the terms of reference, between BNPB, BPBD, between the various government ministries, central, provincial and local government and between sector agencies. Such allocations of roles are necessary to get the necessary cooperation and coordination so that the scarce resources can be used effective and efficiently. Such allocation of responsibilities is meant that all ministries, agencies can contribute to the collective purpose and undertake mitigation measures in areas of their specific concern. For the purpose, a structured disaster planning unit must be established in each government ministries.

- ***Post Audit All Activities related to Non-Engineered Constructions***

As indicated in Chapter 2.2, if disaster risk reduction is to be made a culture, it is high time to do a post audit; there must be an evaluation of the present "disaster management" systems and improvements must be made so that hundreds of people do not die as a result of every natural disaster.

A "post audit" should be introduced concerning all activities related to non-engineered construction. Longitudinal studies should be conducted to document and evaluate various implementation approaches over extended time following implementation. Such study is necessary to identify the specific aspects of a program which are continued without support at the end of the intervention.

As a reminder, the new Disaster Management Law also provides hefty criminal legal sanctions placed on government and civil servants for failure to protect citizens 'pre-, during, and post-disaster' and recognizes international organizations as partners in the new "disaster management" paradigm. The Law also mandates the government to provide compensation for victims of disasters: this potentially enormous recurring cost builds the economic case for the government to ensure more effective disaster risk reduction, mitigation and preparedness. It will be a sincere regret if someday people start proceeding with regards to the reminder above.

- ***Priority of Non-Engineered Construction***

As elaborated in Chapter 3.2.2 and 3.2.2.1, it is evident that non-engineered constructions in Indonesia are vulnerable and therefore, priority is necessary for appropriate measures to protect non-engineered construction. Such measures should include community level programs and economic inputs. It is advisable for disaster risk reduction if there is an interrelated strategy of many parallel approaches.

Technically, a simple solution to solve the problem is by retrofitting these non-engineered constructions using a simple, affordable, replicable method, and suiting the local culture, namely by strengthening the building using wire mesh. This is already explained in Chapter 5. The retrofitting method was applied in various places in Indonesia since 2006 as explained in Chapter 5.3.

- ***Education and Training related to Disaster Risk Reduction***

A long term education program is necessary to prevent recurrence of a similar disaster. As an example, if buildings failed in an earthquake, then it will be necessary to educate architects and engineers about earthquake resistant buildings. Education element is vital in disaster risk reduction measures; however, since the political profile is low, usually it is neglected.

As explained in Chapter 2.1.2, the topic of non-engineered construction has not been introduced in the earthquake engineering syllabus in Indonesia, though most of the buildings damage due to earthquakes is non-engineered constructions. Therefore, structural engineering curriculum in universities must be upgraded and follow the state of the art in modern earthquake engineering and particularly including non-engineered constructions. Equally important is to introduce non-engineered construction in the curriculum of technical high schools. A middle level of technicians must be increased judging from the number of non-engineered constructions in Indonesia. The competence of all parties involved in construction: architects, engineers, surveyors, interior designers, contractors, middle level technicians and also construction workers must be improved.

Research and development in all aspects of disaster risk reduction (risk assessment, planning, protective action, etc.) are also necessary to achieve an effective and efficient implementation.

To achieve an effective disaster risk reduction, training is vital. The government should stimulate training in disaster risk reduction at all levels of society. The training should not only aim at raising awareness, but try to improve the understanding of the disaster process, develop skills and enhance self-realization general principles, but has also to be based on specific local conditions. A multi-sectoral approach is needed to make a disaster risk reduction program effective. Training must be action oriented and demand driven and should focus on vulnerability reduction (Nimpuno, 1992).

The government must be involved through the local authorities, national planning board and ultimately in the national leadership.

The target groups for disaster risk reduction training are: policy makers, national planners, project staff, community groups, NGOs, mid-career engineers, construction workers and trainers which have different training needs at national level as well as regional / local level. Each of the group has specific training needs related to its particular role in the system. All parties must interact as working groups and not isolated. Since each group has its own specific needs, the training methodology and materials must be adapted to these needs.

- Training for **national and local government** is very important since much of the disaster risk reduction work has to be executed by professionals working in local government offices, industry, and education, or for national / international agencies. This group is supposed to coordinate most of the actual disaster risk reduction works. At this level skill development, self-realization and participatory techniques are much needed. International as well as national agencies should be briefed if not trained in local culture, local practice and local wisdom so that all foreign materials can be adapted and adjusted to suit local needs.
- Training for **community level groups** is of the utmost importance and has its own specific needs and methodology. Trainings can be provided by trained government professionals assisted by Universities, local and international agencies; however, in all cases the government shall take the lead. It is suggested for the training and education to include widespread dissemination of “how to do it” manuals and guidebooks on how to build an earthquake safe house and retrofitting. Their training needs include awareness raising, skill development, and strengthening the self-realization capacity.

- Training for **policy makers** cannot be underestimated since policy makers play a major role in developing disaster risk reduction programs. Unfortunately most of the policy makers in Indonesia are not too familiar with disaster risk reduction and learn about disaster related matters from “hear say” only. The training needed by this group is process learning and awareness training. The government MUST improve its human resource management in order there will be a continuation in expertise and thus guarantee the sustainability of the of the disaster risk reduction programs.
- Training for **planners, town planners, agriculture planners, industrial developers** at the national as well as regional level have a big influence in disaster risk reduction. In Indonesia very few developments are scrutinized for disasters since planners are not too familiar with disasters. The main training needs are awareness raising, skill development and process learning.
- Training for **mid-career engineers** is important to increase awareness of earthquake issues, update with the recent developments in earthquake engineering and explain code revisions or regulatory procedures. The standard of earthquake-engineering being taught is important and should be reviewed as an integral part of the longer term earthquake protection strategy.
- Training for **construction workers** is needed to enhance construction skills. The majority of the labor force in the construction industry is filled by the unskilled workers who assist craftsmen on any job, employed and paid per day as casual laborers. Transfer of knowledge and regular training for the unskilled workers are difficult to be held since they are selected by the foreman every day and do not continuously attend at the site. The target for this training shall be the foremen so that they can pass the earthquake resistant construction knowledge to workers that the foremen hired.

  The construction workers are familiar with traditional technologies. Therefore, they need to be upgraded with new building skills using training programs. Training not only encourages self-help, but also provides authorities cost-effective encouragement of improved construction skills and standards.
- The main purpose of a **training of trainers** program (TOT) is to build on professional knowledge of educational staff. A core of trainers can be developed by adding disaster specific information to the normal educational capacities of the trainers. As mentioned earlier, to maintain sustainable trainers, the government should improve its human resource management so that there is continuation and that there is no frequent turn-over of knowledgeable trainers.

- ***Testing for Masonry Constructions***

As mentioned earlier in Chapter 5.10, there is an uncertainty to obtain the properties of masonry wall. Research and development needs with respect to standard material tests to avoid unacceptable approaches and establish a common basis for evaluation and utilization of results. Specimen and test procedure should be made as simple as possible; but within strict guidelines to minimize variability. The testing results should be correlated to full-scale masonry behavior for design, construction, and research purposes.

Although the existing shaking table test result showed that the strength of masonry constructions have increased after the retrofitting technique using wire mesh is applied, further shaking table test should be done in order to make the retrofitting cost lower without reducing the safety of retrofitted buildings. Typical damage of walls due to earthquakes showed that walls without openings cracked diagonally. From these observations, it is not necessary to strengthen the wall using wire mesh covering the whole surface of the wall, but it is good enough to strengthen walls in the diagonal directions only and using ferrocement “beams” and “columns” in place of reinforced concrete practical columns and practical beams, similar to strengthening with ferrocement splints as mentioned in Chapter 5.3.

Until now, the elastic method is good enough to analyze non-engineered constructions. However, with the rapid development in engineering knowledge, software, and computing capability in the future, there is a possibility to analyze the non-engineered constructions using non-linear method that is more practical and easier than currently known.

Appendix A Destructive Earthquakes Dated Back in 1821

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
1	Jepara, Central Java	1821 Dec 25					VII MMI	The earthquake was felt at Jepara and reported as VI-VII as the MMI scale
2	Bulukumba, South Sulawesi	1828 Dec 29					VIII-IX MMI	Bulukumba; in South Sulawesi a destructive earthquake occurred and caused severe damage to buildings; hundreds of people were killed
3	Ambon, Maluku	1830 Mar 28	1h 0m 0s				VII-VIII MMI	Earthquake occurred and caused damage to buildings.
4	Batavia / Jakarta	1833 Jan 28	5h 0m 0s				VII-VIII MMI	Shocks caused damage to buildings and cracked walls. No deaths or injuries were reported.
5	Bengkulu, Sumatra	1833 Nov 24					VIII-IX MMI	A severe earthquake occurred and caused some buildings to collapse or be damaged. A tsunami was observed. No further information.
6	Bogor and Cianjur, West Java	1834 Oct 10					VIII-IX MMI	A violent shock occurred; The earthquake caused severe damage to buildings, some of which collapsed and cracks in the road between Bogor and Cianjur. No deaths or injuries were reported.
7	Padang, West Sumatra	1835 Aug 26					VII-VIII MMI	The earthquake struck Padang and caused slight damage to buildings and cracks in walls.
8	Ambon, Maluku	1835 Nov 01					VII-IX MMI	A large earthquake occurred and caused some buildings to collapse; 60 people injured; landslides in the hills were observed.
9	Mojokerto, East Java	1836 Mar 22					VII-VIII MMI	At Mojokerto, about 60 km west of Surabaya, a shock occurred and caused damage and loss of property.

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
10	Maluku	1837 Jan 21					VII-VIII MMI	Earthquake felt at Saparua, Haruku, and on Nusalaut Island. Damage to buildings and houses.
11	Bima, Sumbawa	1837 Nov 28					VIII-IX MMI	A strong earthquake occurred and caused severe damage to buildings, some of which collapsed.
12	Purworejo, Central Java	1840 Jan 04					VIII-IX MMI	A destructive earthquake occurred at Purworejo and caused severe damage to buildings; two buildings collapsed. Also felt at Semarang, Demak, Solotigo and Kendal on the north coast of Central Java.
13	Ambon	1841 Dec 16					VII-VIII MMI	A moderate earthquake occurred at Ambon. The earthquake was accompanied by a tsunami at Galaga Bay and Buru Island. The tsunami caused damage to some boats.
14	Gunung Sitoli & Baras, Nias Island	1843 Jan 05					VII-VIII MMI	A strong earthquake struck Gunung Sitoli and Baras. The shock was followed by a tsunami, causing damage to some boats. Ground-slump was observed too.
15	Bogor, West Java	1843 May 25					VII-VIII MMI	A shock was felt at Bogor and caused damage to buildings and houses.
16	Cianjur, West Java	1844 Feb 15					VII-VIII MMI	The earthquake hit Cianjur on West Java and caused damage to houses.
17	Menado, North Sulawesi	1845 Feb 08					VIII-IX MMI	A strong earthquake was felt in north Sulawesi and caused the collapse of brick buildings and houses at Menado, Tikala, Tomohon, Tonsarongsong, Tondano and Tanawanko.
18	Teluk Betung, South Sumatra	1852 Jan 09					VII-VIII MMI	An earth tremor was felt in Teluk Betung and caused damage to buildings and houses.

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
19	Kebumen, Central Java	1852 Oct 15					VI-VII MMI	A moderate earthquake was felt at Kebumen. This shock caused cracks in walls at several buildings and houses.
20	Bandanaira, Maluku	1852 Nov 26					VIII-IX MMI	A strong earthquake was felt at Bandanaira-Banda Island and caused some buildings to collapse. The quake was followed by sea waves (tsunamis).
21	Bogor, West Java	1852 Dec 20					VIII-IX MMI	A strong earthquake caused some buildings to collapse.
22	Cirebon, West Java	1853 Nov 30					VII-VIII MMI	A moderate earthquake was felt and caused cracks in walls. No further information.
23	Ternate, Maluku	1855 Jul 14					VIII-IX MMI	A strong earthquake occurred. Severe damage to buildings; one house collapse and 34 people were killed.
24	Semarang, Central Java	1856 Jan 19					VII-VIII MMI	An earthquake was felt at Semarang and caused cracks in walls.
25	Ternate, Maluku	1858 Feb 27					VI MMI	A rather hard shock was felt and caused damaged to walls.
26	Ternate, Maluku	1858 Jun 04					VI MMI	A rather strong earthquake was felt, causing damage to some buildings and houses.
27	Ambon	1858 Nov 09					VI MMI	Some buildings suffered damage by an earthquake.
28	Tondano, North Sulawesi	1858 Dec 13					VII MMI	A moderate earthquake caused 15 sheds to fall. On Ternate, Tidore, Halmahera, Sangihe, Talaud and Banggai islands a tsunami observed.
29	Tulung-agung, East Java	1859 Jul 05					VI MMI	An earthquake occurred and some buildings and houses suffered damage.
30	Halmahera Island, Maluku	1859 Oct 08					VI MMI	At Halmahera, a great number of cottages tumbled down.
31	Tapanuli and Sibolga	1861 Feb 16					VIII-IX MMI	Numerous houses tumbled down. Tsunami was observed at Singkil, Nias and Tello.
32	Buleleng,	1862 Mar 29					VII MMI	A moderate earthquake occurred, causing cracks in walls,

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
	Bali							some of which tumbled down.
33	Karawang, West Java	1862 May 24					VI MMI	A rather severe shock was felt at Karawang, West Java, where walls of some houses were fissured.
34	Bandanaira, Maluku	1862 Sept 15					VI MMI	An earthquake caused cracks in walls
35	Madiun, East Java	1862 Nov 20					VI MMI	Damage to a few buildings caused by a rather severe earthquake.
36	Banyumas, Central Java	1863 Aug 13					VII MMI	The strong earthquake caused heavy damage to a sugar factory.
37	Arfak, Irian Jaya	1864 May 23					VI-VII MMI	The destructive earthquake caused house on Mount Arfak to be set ablaze and some were buried. 250 people killed.
38	Banyubiru, Central Java	1865 Jul 17					VII MII	Some buildings and houses suffered considerable damage caused by an earthquake.
39	Ambarawa, Central Java	1866 Apr 22					VI MMI	Because of an earthquake, walls of some houses and barracks were fissured.
40	Yogyakarta, Central Java	1867 Jun 10					VIII-IX MMI	In Yogyakarta and Surakarta 372 houses collapsed or partially collapsed, while only 5 persons lost their lives.
41	Ternate, Maluku	1867 Nov 03					VI MMI	A moderate shock caused fissures in the walls of numerous houses.
42	Banyumas, Central Java	1871 Mar 27					VI MMI	Fissures in the walls of government buildings and houses, which were caused by an earthquake.
43	Bengkulu, Sumatra	1871 Aug 18					VI-VII MMI	The quake caused some houses to tumble down in Bengkulu and Tebingtinggi
44	Salatiga, Central Java	1872 Oct 10					VI MMI	A rather strong shock felt at Salatiga caused fissures in walls.
45	Ciamis, West Java	1873 Feb 05					VI MMI	The walls of numerous buildings were cracked.
46	Mandailing, N.Sumatra	1873 Aug 19					VI MMI	Many houses were damaged, due to earthquake.
47	Tapanuli,	1873 Oct 07					VI MMI	This quake caused damage to some houses and bridges.

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
	North Sumatra							
48	Kuningan, West Java	1875 Oct 25					VII-VIII MMI	The quake was felt at Kuningan, Sumedang and Manonjaya. 628 houses damaged and seven people killed.
49	Kajeli, Ceram	1876 May 28					VII MMI	A few houses suffered damage and a mosque tumbled down at Kajeli-Ceram Island-Maluku.
50	Kudu, Central Java	1877 Feb 21					VI MMI	A rather strong shock, felt at Kedu and Wonosobo in Central Java, caused damage to several buildings.
51	Pasuruan, East Java	1889 Nov 04					VI MMI	The quake caused crack in walls.
52	Negara, Bali	1890 Jul 11					VII MMI	The earthquake caused three pillars of the Justice Buildings to split horizontally and walls to tumble down.
53	Bandanaira, Maluku	1890 Nov 23					VII MII	Damage to most of the houses and buildings.
54	Pati, Central Java	1890 Dec 12					VIII MMI	This quake also felt at Juwana; caused many houses to fall. Several people killed and injured.
55	Prapat	1892 May 17					VI MMI	The shock caused severe damage to three buildings.
56	Timor Island	1896 Apr 18					VII-VIII MMI	The quake was also felt at Alor Island; 250 people killed and most of the settlement damaged.
57	Jumajang, East Java	1896 Jul 01					VI MMI	The walls of some houses were split.
58	Wlingi, East Java	1896 Aug 15					VII MMI	At Brangah-Wlingi, many public and private buildings/houses damaged.
59	Tulung-agung	1896 Aug 20					VII MMI	The shock caused severe damage to several Chinese houses.
60	Ambon, Maluku	1898 Jan 17					VII MMI	Many houses were destroyed by this quake.
61	Sukabumi, West Java	1900 Jan 14					VII MMI	Felt over Priangan, Bogor and Banten. Most damage to stone houses occurred at Sukabumi, but no lives were lost.
62	Lais,	1902 Aug 27					VI MMI	Fall of plaster and cracks developed in walls.

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
	Bengkulu							
63	Sedayu, East Java	1902 Aug 31					VI MMI	Ground-slumps were observed, walls were disturbed. A series of aftershocks felt during the period 26 Sep - 9 Oct, the heaviest one in August 31, accompanied by a roaring sound.
64	Bandanaira, Maluku	1903 Feb 14					V MMI	Suspended objects swung and moveable objects were thrown down.
65	Banten, West Java	1903 Feb 27					VI MMI	This quake felt over Banten; small cracks developed in walls.
66	Siri-Sori, West Sumatra	1904 Jul 05					VIII MMI	A part of the pier was destroyed and sailing boats on the coast of Siri-Sori sank as a result of the high waves.
67	Lemo, Central Sulawesi	1907 Jul 30					VIII MMI	Destructive at Lemo, where 164 houses and 49 rice-warehouses collapsed; shocks were frequently felt until August 2. Damage to buildings was also done to Colo, Anja, Olu Congko and Paku.
68	Atapupu, Timor Island	1908 Mar 24					VII MMI	The quake strongly felt at Atapupu in Northeast Timor. Cracks developed in wall of a fortress; a part of the wall fell. Damage to buildings was also done in the Chinese blocks. Cracks developed in the beach sands about 25m long. A tsunami was observed.
69	Rajamandala, Cianjur	1910 Dec 18					VII MMI	Cracks developed in walls at Rajamandala-Cianjur-West Java.
70	Campaka, Sukabumi	1912 Jan 21					VI MMI	Cracks developed in walls at Campaka-Sukabumi, West Java.
71	Japen Island, Irian Jaya	1914 May 26					IX MMI	All brick buildings collapsed on Japen Island, Ansus & Pom were affected by tsunami. A few people died.

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
72	Kepahyang, Bengkulu	1914 Jun 26					IX MMI	All stone houses suffered severe damage. None of the many wooden houses sustained damage. Twenty persons were killed and 20 injured. Roads and bridges were destroyed. Damage was also done at Lais, Manna and Seluma.
73	Madiun, East Java	1915 Dec 01					VIII MMI	Nearly all buildings in the Sudono sugar estate were cracked. The chimney of the sugar factory topped down. A certain amount of damage was also done at Maospati and Magetan.
74	Maos, Central Java	1916 Sept 09					IX MMI	Most destruction took place in around Maos. About 340 brick stone buildings collapsed completely and many others were damaged at Maos and Kasugian. Cracks developed in walls, ground-slumps were reported. A few mud or sand craters were formed where jets of water spurted through holes or fissures, causing people to panic. Four hundred houses collapsed in the Selarang district. Damage to structures and cracks in the ground were also found in various places. School buildings were among those most generally and severely damage, due in considerable part of unsuitable design for resistance to shaking. The major destruction, however, was in a more thickly settled district, where unfavorable geological conditions and poor structural work increased the damage.
75	Bali	1917 Jan 21					IX MMI	Ground slumps and ground-slides were observed at various places. Many houses suffered damage and about 1500 people were killed due to ground-slides.
76	North Irian Jaya	1919 Nov 21					VIII MMI	Some damage was caused by a strong quake in the eastern part of North Irian Jaya. A few houses collapsed. Earth fissures developed in the ground and walls were disturbed.

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magnitude	Intensity	Description
77	Ambon, Maluku	1920 May 10		1120 km south of Banda and Irian Jaya			VI MMI	Cracks in walls reported at Ambon, Saumlaki and Banda. The quake itself had its origin about 1120 km south of Banda and Irian Jaya.
78	Tapanuli, North Sumatra	1921 Apr 01		a narrow belt aligned NW-SE running for some 80 km from Pangurusan to Tarutung			IX MMI	Epicenter tract occupied a narrow belt aligned northwest-southeast running for some 80 km from Pangurusan to Tarutung. The area worse hit was the region southwest of Lake Toba. Buildings and bridges collapsed at Sipoholon; furthermore ground-slides and ground-slumps were reported. The quake also felt as far as Sabang, Penang, and Gunung Sitoli.
79	Sangkulirang, East Kalimantan	1921 May 14					VIII MMI	Damage at Sangkulirang and more intense in the islands of Rending, Kariorang and Sekuran. Houses collapsed and gaping fissures were observed. The shock was associated with a tsunami which swept the sea, causing considerable damage at Sekuran.
80	Sentani, Irian Jaya	1921 Oct 10					VII MMI	A major earthquake was felt as far as Dobo, but was destructive around Lake Sentani in south-east Irian Jaya. Ground-slides, boulders, and a large mass of limestone damaged up a branch of the river temporarily in the village of Doormantop.
81	Tarakan, East Kalimantan	1923 Apr 19					VIII MMI	The earthquake was recorded by sensitive seismographs all over the world. The shock was strongly felt at Tarakan about 140 km north of the center and followed by a number of aftershocks. Brick buildings collapsed, cracks developed in the ground, and streams were affected. The kitchen of a house seemed to be displaced over a distance of about 1 m toward the west. Structures on solid ground suffered little damage.

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
82	Banten, West Java	1923 May 12		7.3ºS, 105.8ºE			VII MMI	The shock was felt over West Java and South Sumatra as far as Krue. Damage was done at several places; at Pelabuhanratu a water tower was thrown down.
83	Maos, Central Java	1923 May 15		7.7ºS, 109.2ºE			IX MMI	The shock was felt intensively over western Central Java. Destructive effects were particularly pronounced in Maos.
84	East Kalimantan	1924 Apr 13		0.3ºN, 118.2ºE			VII MMI	The tremor was generally felt at several places in East Kalimantan and as far north as the island of Tarakan; it was followed by aftershocks. As a result of the main shock, seven houses collapsed.
85	Central Java	1924 Nov 12		7.3ºS, 109.8ºE			VIII-IX MMI	The center was located in a mountainous region. Damage was generally caused by ground-slides.
86	Wonosobo, Central Java	1924 Dec 02		7.3ºS-109.9ºE			IX MMI	The quake seemed to be preceded by foreshocks. Destructive at Wonosobo and damage was also done to stone buildings outside Wonosobo. Approximately 2,250 houses collapsed and in some villages most damage was caused by ground-slides. Altogether about 727 people were killed. The quake loss was estimated by the local authorities at about 61,000 guilders.
87	Tarakan, East Kalimantan	1925 Feb 14					VII MMI	The exact origin was unknown, but the shock was strongly felt at Tarakan and Lungkas and it was preceded by a rumbling sound.
88	Bacan Island, Maluku	1925 Jul 24					VII MMI	Exact origin unknown. Strongly felt at Labuhan (Bacan Island), accompanied by a roaring sound; cupboard overturned, a pendulum clock fell.
89	Singkarak, West Sumatra	1926 Jun 28		0.7ºS, 100ºE			VIII-IX MMI	Destructive around Lake Singkarak; Sijunjung, Muarabungo & Alahan Panjang suffered damage. The quake was followed by a train of aftershocks & generally felt intensively over West Sumatra and in particular at Padangpanjang. A part of Lake Singkarak subsided, and many people were injured.

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
90	Prupuk, Central Java	1926 Dec 13					VIII-IX MMI	Destructive at Prupuk and Margasari; minor damage at Dubuktengah, Kaligayan, Wonosari, Danurejo, Jembayat, Pakulaut and Kalisosok. A few people were injured.
91	Donggala, Central Sulawesi	1927 Dec 01		05°S, 119.5°E			VII MMI	A major earthquake caused damage to buildings at Donggala, Borowaru and neighbouring places. Earth fissures and subsidence in the ground were reported. The damage was mainly confined to the Palu Bay area. A tsunami was observed that caused serious loss of life and property in coastal villages. About 50 people sustained injuries and 50 died.
92	Bali	1930 Apr 27					V MMI	Exact origin was unknown. Damage was done in south Bali by a moderate tremor. Walls were cracked at Denpasar and Tabanan, earth fissures in the ground occurred at Benoa. The shock was also felt over East Java.
93	Bumiayu, Central Java	1931 Jan 21		7.3°S, 108.9°E			VIII MMI	In general, damage was confined mostly to older structures or buildings of poor materials and poor construction.
94	South Sumatra	1931 Sept 25					VII-VIII MMI	Felt over South Sumatra and West Java and as far west as Padang. Foundations of most buildings subsided. Difficult to walk as a result of the earthquake. In Kalimantan a rumbling sound was heard.
95	Tondano, North Sulawesi	1932 May 14		0.5°N, 126.0°E			VII MMI	The strong earthquake was felt as far north as Mindanao. The major destruction took place at Kakas, south of Lake Tondano; 592 houses collapsed, 115 people sustained injuries and the death of six people was reported. Damage was also done at Langowan, Poso, Tondano, Waluyama, Rembohan, Koya and Lekupang. Ternate in north Maluku suffered minor damage. On the coast between Amurang and Tompoan vertical gaping cracks developed in the beach sands and the sea side of the cracks sagged.

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
96	Seram, Molluce	1932 Sept 09		3.5°S, 128.3°E			VII MMI	The origin seemed to be in Tolehu Bay. A few old buildings collapsed at Wae and Tolehu. Ground-slumps & ground-slides occurred.
97	South Sumatra	1933 Jun 25		5.0°S, 104.2°E			VIII-IX MMI	The quake seemed to have been followed by a number of aftershocks. Damage to structures over the western part of south Sumatra. Gaping fissures and subsidence in the ground were observed along an imaginary line connecting Kotaagung with Makaka, crossing the Barisan Mountain range (Bukit Barisan).
98	South Tapanuli, Sumatra	1934 Sept 21		1.0°N, 99.0°E			VII MMI	Generally felt violently. Pendulum clock stopped; doors and windows rattled. Cracks developed in walls; roofs of some houses ruined.
99	Batu Island, North Sumatra	1935 Dec 28		0.3°S, 97.9°E			VII-VIII MMI	Damage on Batu Island. Two mud islets, Bola and Sigata, were thrown up by the shock. At Padang cracks developed in walls. Trees and telephone poles swayed. A few buildings collapsed at Sibolga.
100	East Java	1936 Mar 01					VII MMI	Exact origin unknown. Damage was generally done in Central and East Java. The shock was also felt over Bali and southeast Kalimantan.
101	Sangir, North Sulawesi	1936 Apr 01		3.6°N, 126.7°E			VIII-IX MMI	The quake seemed to be followed by numerous aftershocks. Destructive in the Sangir-Talaud Islands. Approximately 127 houses collapsed. Cracks in walls at Lerung.
102	Banda Aceh	1936 Aug 23		6.1°N, 94.7°E			VII-VIII MMI	The quake was strongly felt at Banda Aceh, Lhok Sukon, and Lhokseumawe and was followed by a number of aftershocks. Caused damage to buildings; as a result of the shocks of 9 people said to have perished, 20 people were badly injured.
103	Tapanuli, North Sumatra	1936 Sept 9		3.5°N, 97.5°E			VIII MMI	The quake caused minor damage at Medan and it was felt as far east as Malaysia. The most destructive effects of the quake were confined to the Karo region; 17 people were

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
								killed due to landslides in the hills. Numerous cracks appeared in the ground between Katacane and Kabanjahe. A certain amount of damage at Parapat, Brastagi, and Tanjung Putri. Cracks developed on walls at Langkat.
104	Sanana, Molluca	1936 Oct 19		2.0ºS, 126.0ºE			VIII MMI	Aftershocks were also felt in Sula Island. Moveable objects were overthrown and a rumbling sound was heard. At Sanana 24 houses collapsed, great fissures appeared in the ground close to the market. At Wai Ipa 14 houses were damaged & at Wailau 2 buildings sustained damage.
105	Tapanuli, North Sumatra	1936 Oct 27		0.2ºS, 98.8ºE			VII MMI	The quake was felt over Tapanuli, West Sumatra and also locally in East Sumatra. At various places slight damage to structures and ground-slumps were reported.
106	Yogyakarta, Central Java	1937 Sept 27		8.7ºS, 110.8ºE			VIII-IX MMI	Felt as far east as eastern Lombok. In general, south Central Java was badly damaged and slight cracks in walls were found in East Java. The region of greatest destruction was in Yogyakarta Province. At Klumpit one house was torn apart, one person reported killed. At Prambanan 326 brick/stone houses collapsed. At Klaten 2,200 houses sustained damage; at various places underground pipelines were broken.
107	Banda, Maluku	1938 Feb 02		5.0ºS, 131.5ºE			VII MMI	The shock was felt on the Banda and Kei islands. At Tual glassware was broken, a pendulum clock stopped. On Banda Island and also on Kei Island great damage was caused by tsunamis.
108	Tomini Gulf, Central Sulawesi	1938 May 20		0.7ºS, 120.3ºE			VIII-IX MMI	The tremor was felt as far west as east Kalimantan and as far north as Gorontalo (Minahasa). The shock was associated with a tsunami which swept the sea, causing serious loss of life and property; 942 houses collapsed and a few persons were drowned.
109	Unknown	1938 Aug 02					VII MMI	Exact origin unknown. Poorly built structures were badly damaged; in the mountainous regions, more damage was

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
								done due to landslides.
110	Bengkulu, Sumatra	1938 Aug 18		3.8ºS, 102.8º			VII MMI	The shock was felt over West Sumatra, Palembang, and Bengkulu and on the Mentawai Islands. Fall of plaster and cracks in walls were reported at some places in Bengkulu.
111	Flores	1938 Oct 20		9.2ºS, 123.2ºE			VII MMI	Walls were badly cracked in Flores. Ground-slides at Larantuka. The quake seemed to have aftershocks.
112	Bali	1938 Oct 30		8.9ºS, 115.8ºE			VII MMI	As a result of the shocks, cracks appeared in walls and the principal mosque was badly damaged at Sakara.
113	Central Java	1939 Jun 27		6.9ºS, 108.5ºE			VII MMI	Fall of plaster and small cracks in walls in the Cirebon Residency. More damage was done at Sodomantra, Jepara and Manis Kidul.
114	East Java	1939 Aug 11		6.5ºS, 112.4ºE			VII MMI	Rembang and Surabaya were rocked; suspended objects swung. A brick building collapsed at Brondong.
115	Central Sulawesi	1939 Dec 22		0.0ºS, 123.0ºE			VIII MMI	This major earthquake was felt over north and central Sulawesi, East Kalimantan and as far north as the Sulu Islands. Cracks developed in walls at Gorontalo and at Langonan; cupboards overturned and a few people were injured. Houses collapsed at Kalo. Luwuk, Labuha and on the Sula Islands. At Mandar and Meulaboh in Central Sulawesi houses were shaken.
116	Tapanuli	1941 Oct 11		0.6ºN, 97.6ºE			VII MMI	Strongly felt over Tapanuli; slight damage was done at Sibolga.
117	Gorontalo	1941 Nov 09		1.4ºS, 121.1ºE			VIII MMI	Brick/stone buildings collapsed at Gorontalo, Paleleh and Cibawa. Ground-slumps and landslides in the hills were reported.
118	Yogyakarta, Central Java	1943 Jul 23		8.6ºS, 109.9ºE			VIII MMI	The disturbance was most intense along the south coast of Central Java, between Garut and Surakarta, a distance of about 250 km. The deaths of 213 people have been reported and about 2,096 persons were seriously injured; approximately 2,800 houses were damaged.

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
119	East Java	1950 Jun 19	19h 36m 54s	6.2ºS, 112.5ºE			VII MMI	Destructive in and around Gresik. Felt slightly in South Kalimantan and as far west as West Java
120	Bima, Sumbawa	1954 Nov 02	16h 24m 54s	8.0ºS, 119.0ºE		6.75	VII-VIII MMI	Felt over South Sulawesi, Lombok and Flores. Ground-slumps and rock-slides were caused by the earthquake in northeast Sumbawa. Bima and Raba suffered most damage. Nearly all brick/stone houses were cracked and some collapsed completely. Two buildings made of reinforced concrete did not sustain any damage. The pier of Bima harbor was bent outward. A small customs-house seemed to be displaced over a distance of about 0.5 m. No lives were lost.
121	Malang, East Java	1958 Oct 20	08h 12m 30s	9.5ºS, 112.5ºE	100	6.7	VII-VIII MMI	Seriously damaged houses in the Malang area. Earth fissures at various places and land-slides in the mountainous regions. Eight persons lost their lives.
122	Una-Una, Central Sulawesi	1960 Apr 29	17h 16m 20s	0.5ºS, 121.5ºE			VII-VIII MMI	Felt over North and Central Sulawesi. Destructive on the Una-Una Islands. No lives were lost.
123	Tulung-agung, East Java	1960 Oct 11	04h 44m 40s	8.0ºS, 112.5ºE			VI-VII MMI	The quake was strongly felt at Tulungagung, where people were awakened by creaking of buildings and where plaster cracked and fell. The shocks was felt as far west as Baturetno in Surakarta and as far east as Tanggul in Besuki. This earthquake was widely felt in southern East Java over an area of about 15,000 km^2.
124	Flores	1961 Mar 17	02h 21m 04s	8.1ºS, 122.3ºE			VII-VIII MMI	Damage in most villages in Central Flores; one person killed.
125	Campur darat, East Java	1961 May 07	11h 32m 05s	8.5ºS, 112.0ºE			VI-VII MMI	Damage to brick buildings at Campur darat and Kebonagung Tulungagung. Reports indicated that the macro seismic area extended as far west as Banyumas, Central Java and as far east as Besuki in East Java. Evidence indicated that the tremor had a maximum intensity of VII in the immediate vicinity of the center. The shock also felt at

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
								Jatisrana - Surakarta, Klaten, Maos, Malang and Klakah. To the north the macro seismic area was limited by the mountain range Kendeng; however some places, such as Demak and Watubelah to the north of the mountain range, felt this tremor as Intensity II. The great macro seismic extent suggested that the quake was deep-seated. In the vicinity of the center slight damage was caused to old structures made of bricks bonded with lime mortar.
126	Wlingi, East Java	1962 Dec 21	07h 44m 19.7s	9.0°S, 112.2°E	64		VI MMI	Cracks in walls in southern East Java. The shock was felt as far east as the Island of Bali. A moderate tremor was felt in Wlingi and neighbouring places in Kediri and also felt in most places in the Madiun area. A large number of people in buildings in Madiun and Kediri felt the quake and its intensity was high enough to cause some panic in public places, such as schools and markets. Macro seismic and instrumental data indicated that the tremor was of tectonic nature and deep-focused.
127	Ponorogo, East Java	1963 Jun 27	18h 46m 58s	8.3°S, 112.2°E	180		IV-V MMI	The earthquake caused slight damage in Ponorogo. The shock was felt in central and eastern Java. The western-most place was Yogyakarta and to the east, Besuki, which detected the tremor; both places reported Intensity II.
128	Labuan, West Java	1963 Dec 16	09h 45m 35s	6.2°S, 105.4°E	55	5.0	V MMI	The earthquake caused slight damage in Labuan where cracks developed in walls. A large number of people felt the quake and its intensity was high enough to cause some panic among the people in Jakarta; however no damage was reported by this shock. The seismograph of the Meteorological Service was out of order. The shock was felt as far east as Tasikmalaya as Intensity II and as far west as Kotabumi in South Sumatra as Intercity II. The shock was also felt in most places in Priangan as Intensity II-III.

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
129	Banda Aceh	1964 Apr 02	08h 11m 55s	5.9°N, 95.7°E	132	5.2	VII MMI	The earthquake was the strongest shock ever recorded in this region since that of August 23, 1936 which caused considerable damage to buildings. The quake was most strongly felt at Banda Aceh where the intensity was high. About 30-40% of the brick buildings sustained damage. The village worst hit was Krueng Raya.
130	North Maluku	1965 Jan 25	21h 02m 51.4s	2.6°S, 126.1°E	33	6.3	VII MMI	A rumbling sound was heard at Sanan Coastal villages destroyed and 5 persons reported killed, due to a tsunami.
131	Tapanuli, North Sumatra	1965 Jul 25	10h 40m 40.4s	2.0°N, 99.3°E	98	5.3	VII MMI	At Sarula and Onang Hasang in Tapanuli the intensity reached VII MMI; damage to brick buildings and ground-slumps were observed.
132	Malang, East Java	1967 Feb 20	05h 12m 55s	8.5°S, 113.5°E	80	6.2	VIII-IX MMI	The place worst hit was Dampit, a district situated just to the south of Malang; according to questionnaire reports, 1539 buildings were wrecked, 14 people were killed, 72 people were injured. Next to Dampit was Gondang where, according to a report, 9 people were killed, 49 people were injured, 119 buildings collapsed completely and another 402 buildings were cracked. 5 mosques were ruined. Attention should also be drawn to Trenggalek where 33 wooden houses were reported cracked and some houses have been moved slightly. In Besuki, the easternmost district of East Java, the intensity was of the order of III to VI MMI; in Tanggul, buildings sustained slight damage only. The shock was felt to the west as far as Banyumas (Cilacap); to the north a chain of hills in western East Java form a sort of barrier to the propagation of seismic waves. No report was received about a tsunami.
133	Tinambung, South Sulawesi	1967 Apr 11	13h 09m 11s	03.7°S, 119.3°E	51	4.9	VII-VIII MMI	The tremor was felt over a wide area. The area worst hit covered the coastal lowlands extending from Campalagian to Tinambung. A high tsunami was generated during the

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
								main shock, causing serious loss of life and property in coastal villages; 58 people were reported killed by the collapse of brick buildings, about 100 people were injured, 13 people were drowned in the sea or missing. Fissures were local in nature and might have been due to the loose formation of the soil in that area.
134	Aceh, North Sumatra	1967 Apr 12	11h 51m 50.2s	05.3°N, 97.3°E	55	6.1	VII MMI	The earthquake was felt mainly over the eastern coastal areas of Aceh, being located on alluvial deposit and to the south as far as Kisaran. Farther inland the shock was felt in Takengeun, situated in the mountainous region. No report was received from places on the west coast of Aceh. The maximum intensity, about midway between Lhoseumawe and Sigli, probably did not exceed VIII MMI. The places worst hit were Jeunieb, Pendada and Jeupa Bireun. Damage was done to 5 mosques, 59 brick and wooden houses which were used for religious purposes, 11 schools buildings, 5 bridges and about 2000 brick and wooden dwelling houses. Furthermore, earth-slumps, rock-slides, cracks and fissures were observed over a wide area; mud and sand erupted from fissures in soft, water-saturated deposits at some places. In Sigli the quake was followed by an enormous tsunami. Eastern Aceh is a region of only moderate earthquake activity as compared, for example, with the west coast of Sumatra.

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
135	Tambu, Central Sulawesi	1968 Aug 15	06h 14m 15s	0.7°N, 119.8°E			VII-VIII MMI	Generally along the west coast of the northern part of central Sulawesi an increase of high waves was noticed shortly after the main shock, in particular in Tambu Bay; the wave height in Tambu was about 8 to 10 metres and might have reached some coconut-tree tops; the waves swept further inland, to about 100-300 metres from the coast (Tsunami). Most of the beach of the inner part of Tambu Bay is under sea water; slumps with surface trace of faulting and hot springs in several places were observed. Loss of life and considerable damage were chiefly caused by the tsunami along the coast of Mapaga (200 people killed and missing, 790 wooden houses wrecked.) In Tambu 7 wooden houses on pillars have been moved in a northwesterly direction. In Sabang a roaring sound was reported. The small island of Tuguan is uninhabited and is still intact. The main shock seemed to be followed by aftershocks and one of those was shallow (11 km). A report from Central Sulawesi about a sound that was also heard in Tambu on October 19, 1968 would strengthen the report of the presence of the shallow earthquake which was detected at 0.1°N-119.8°E in Mapaga Bay
136	Majene, South Sulawesi	1969 Feb 23	08h 36m 55.6s	03.1°S, 118.5°E	13	6.1	VIII MMI	This quake killed 64 people, 97 others were injured and about 1,287 man-made structures were wrecked including mosques which completely collapsed due to poor construction. The place worst hit was Majene. Eighty percent of brick buildings sustained serious damage; some of them completely collapsed. The pier of the harbor was cracked in several places possibly due to a subsidence of the submarine surface just outside the harbor; gaping cracks, about 50 m long, caused three brick buildings serious damage; the center market

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
								completely collapsed resulting in several deaths and severe property damage. Campalagian and Wonomulyo, located on alluvium and respectively about 30 km and 50 km east of Majene, also sustained structural damage. Generally wooden houses were able to resist the shaking and much of the damage there was caused by the collapse of unreinforced concrete walls. A tsunami generated by the quake struck the coastal villages north of Majene. The wave height reached about 4 m at Paletoang and 1.5 m at Parasanga and Palili. The construction in the villages was principally wood frame and, due to their location at the end of a bay, the wooden houses were swept away the tsunami. In these coastal places banana trees were almost totally destroyed. Damage to the mosque was probably due to the fact that the old structures were made of brick without reinforcing iron rods. Several bridges were damaged in this narrow lowland plain. Between Somba and Parasangka great blocks of Neogene marls and tuffs tumbled down and in some places the road was buried by these blocks. Also from the edge of the raised coral reefs greater and smaller parts were loosened and tumbled down onto the beach. Fissures were also observed at several places. People interviewed said a roaring sound was heard coming from the sea. The shock was felt as far south as Ujung Pandang.
137	Sukabumi, West Java	1969 Nov 02	18h 53m 06.6s	06.5°S, 107.1°E	57	5.4	V MMI	A relatively strong earthquake was felt in southern West Java. In the Bogor area suspended objects swung as a result of the shock; in Campaka, where the intensity was highest, the only known structural damage was cracks

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
								produced in the walls of some badly constructed buildings. The shock was also slightly felt in Jakarta. In the south Bogor area, an aftershock seemed to be felt one hour later. In Sukabumi a poorly constructed brick building was reported collapsed as a result of this earthquake.
138	Sentani, Irian Jaya	1971 Jan 10	16h 17m 03.7s	03.1°S, 140.1°E	normal	7.3	VI-VII MMI	The quake was felt in most places in northern Irian Jaya and rocked Jayapura and Sentani. In Jayapura cracks developed in walls of brick buildings and ten wooden buildings on pillars floating on water collapsed completely. In Sentani, 40 km away from Jayapura, a church was cracked and at 10 km further inland about 14 wooden houses on pillars toppled. In Genyem, about 40 km away from Sentani, earth-slumps and fissures which erupted mud and sand were observed. A sound was heard like gun fire. This area is sparsely inhabited and the sketchy information is due to difficult communications.
139	Sibolga, North Sumatra	1971 Feb 04	22h 33m 22.0s	0.6°N, 98.8°E	normal	6.3	V-VI MMI	Damage to brick buildings developed in Pasaman, Natal, Panangsore, Sibolga and Pasir Ulu estate. Fissures were observed in Sibolga and hot springs developed in Tarutung. The shock was generally felt in various places in the eastern part of north Sumatra and as far east as Singapore. No loss of life was reported from this earthquake.

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
140	Bantar Kawung, Central Java	1971 Jun 16	14h 44m 22.5s	7.2°S, 109.1°E	35	5.2	VII-VIII MMI	The shock was generally felt in western Central Java. The place worst hit was Buaran, about 6 km west of Bumiayu; further, in Bantar Kawung and Jipang, respectively some 12 and 17 km west of Bumiayu, most brick buildings suffered considerable damage. In the affected area 1,377 buildings sustained damage; wooden houses generally resisted shaking but some poorly constructed buildings slanted toward east or west and some collapsed completely. Despite this only one person was reported killed and 6 injured. The damage might have been due to old structures made of brick which are not well cemented and are without reinforcing iron rods. The unconsolidated river deposits may largely be responsible.
141	Mamuju, South Sulawesi	1972 Sept 06	15h 00m 25.3s	2.5°S, 119.1°E	36	5.8	IV MMI	The quake rocked the Mamuju area in the northwestern part of south Sulawesi. Only slight damage to brick buildings resulted from the tremors. The quake was preceded by a roaring sound similar to that of a bomb. The shock was felt as far south as Majene.
142	East Java	1972 Nov 04	21h 36m 54.0s	08.4°S, 112.2°E	126	6.0	V-VI MMI	Southern Blitar-Trenggalek area experienced an earthquake at 04h 36m L.T. in the morning. Gandusari reported a fairly strong shock as Intensity V-VI MMI. The tremor caused cracks in the walls of brick buildings and a great number of people were awakened from sleep. The shock was felt as far as the Yogyakarta-Surakarta border and this far-extended felt area strengthened the view that shock was deeper than normal.

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
143	Sumedang, West Java	1972 Dec 19	21h 47m 00s	06.9ºS, 107.8ºE	very shallow	4.5	V MMI	At 21h 47m L.T. On December 19th 1972 the Sumedang area experienced a tremor of moderate strength which was slightly shallower than normal, as indicated by the small area affected. The quake caused slight damage to old brick buildings and panic among the people. In Cibunar, Rancakaleng and Pasaribu villages the intensity was IV MMI. In Sindang village the same quake was felt as V MMI; ground-slides and ground fissures were observed.
144	Pelabuhan-ratu, West Java	1973 Nov 26	15h 51m 12.8s	06.8ºS, 106.6ºE	62	4.9	V MMI	In Pelabuhanratu, the quake was felt as Intensity III-IV MMI. At Citarik and Cidadap villages, where the intensity was highest, the only known structural damage was slight cracks produced in the walls of old brick buildings and falling of plaster. Ground cracks and ground-slides were observed.
145	Siau Island, North Sulawesi	1974 Feb 27	08h 21m 57.7s	2.7ºN, 125.4ºE	normal	5.2	V MMI	The quake was followed by an aftershock on March 13, 1974 occurred a shallow earthquake again off the west coast of Siau Island. The shock caused people to panic; due to the continuous strong shocks and the loose formation of the soil in that area, it caused ground-slides, ground cracks and damage to buildings.
146	Banten, West Java	1974 Nov 10	02h 10m 55.2s	6.5ºS, 105.3ºE	51	6.1	VI MMI	This quake caused people to awaken. In Leuwiliang, southern Banten, one stone building collapsed and cracks developed in the walls of some houses. The shock was felt as far as Lampung and Pringsewu in South Sumatra and also in Jakarta by some people.
147	Banda Island, Maluku	1975 Jan 15	17h 42m 24s	5.0ºS, 130.0ºE	normal	5.9	VII MMI	Heavy damage at Bandanaira, 81 houses seriously damaged, 4 houses moderately damaged and 2 houses slightly damaged. The earthquake was followed by a tsunami.

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
148	Sanana-Sula Island	1975 Mar 05	08h 22m 23s	02.4°S, 126.1°E	normal	6.5	VI MMI	The shock felt by many people in Sanana-Sula Island at 01.30 UTC for 8 seconds and at 02.24 UTC for 1.5seconds. Cracks in walls and plaster falling. The earthquake was followed by a tsunami. The height of sea water was about 1.20 m; it reached the road in Sanana and caused people to panic. No person killed.
149	Kupang,Timor Island	1975 Jul 30	16h 17m 11.6s	9.9°S, 123.9°E	30-50	6.1	VII MMI	The walls of many houses fell, cracks in walls, plaster fell. No damage to buildings or houses of good construction. The earthquake was followed by a sound like thunder from the ground.
150	Purwokerto	1976 Feb 15	03h 31m 49s	7.2°S, 109.3°E	22	5.6	IV MMI	Almost everyone was awakened from sleep due to the earthquake and the sounds from the buildings/houses. The shock was also felt at Ajibarang, Kedungbanten, Tegal, Brebes, Pekalongan, Magelang and Semarang. No damage reported.
151	Kotacane, Aceh	1976 Jun 21	03h 56m 31.7s	3.2°N, 96.3°E	33	6.1	VII MMI	Aceh - Cracks occurred in the walls of the local government office building. Sibolga - cracks in walls of the power-house building at Pinagsore Airport. The earthquake was also felt by many people in Banda Aceh and Medan, but no damage was reported.
152	Jayapura	1976 Jun 26	08h 18m 29s	3.2°S, 12.8°E		6.8	IV MMI	The shock was felt by many people but no damage was reported. According to the newspaper report, the earthquake caused a landslide and ground cracks in the hinterland of Irian Jaya.
153	Bime, Epiomek, Nalca and Okbad in Irian Jaya	1976 Jun 26	03h 18m 55.5s	4.6°S, 139.8°E	33	7.0	VIII MMI	Severe damage caused by this earthquake. Moderate or slight damage occurred in Langda, Ambon, Japil, and Oksibil. Due to lack of transport and communications no complete report available.

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
154	Seririt and Busungbiru	1976 Jul 14	15h 13m 22s	8.2°S, 114.9°E	normal	6.2	VIII MMI	90% brick buildings and houses collapsed. In Tabanan and Jembrana more than 75% buildings and houses severely damage. 559 people killed, 850 people seriously injured and some than 3200 injured.
155	Nalca	1976 Oct 29	10h 51m 01s	4.7°S, 140.2°E	30-50	6.0	VIII MMI	Bime - 62 people killed. Langda - 46 people killed. The wooden houses of native people, which were built in the slope, collapsed and were buried by a landslide, but the wooden houses which were built on the flat ground suffered no damage.
156	Bangli, Bali	1977 Jan 26	21h 11m 29.5s	8.25°S, 115.3°E	normal	5.0	VI MMI	This relatively strong quake was felt in Bangli and surroundings. In Kayubihi village cracks were produced in the walls of one semi-permanent school building; a monument and temple collapsed. About 90% of houses damaged. In Banjar Antugan Jehem village more than 80% of buildings and houses cracked. Ground-slides and cracks in the ground were observed in the Melangit River, about 500 m eastward of Kayubihi and Antugan. No deaths or injuries reported.
157	Pasaman, West Sumatra	1977 Mar 09	06h 17m 29s	0.4°N, 99.7°E	normal	6.0	VIII MMI	In Sinurat, the quake caused serious damage to 737 houses, one market, 7 school buildings, 8 mosques and 3 office buildings. In Talu 245 houses, 3 school buildings and 8 mosques also damaged. Almost all wooden houses in that area were slanted and shifted from their foundations. Cracks in the ground 5-75 cm wide were observed. Felt III MMI at Padang and Pandangpanjang. No deaths reported.
158	Sumbawa, East Nusa-tenggara	1977 Aug 20	03h 06m 08s	11.1°S, 118.5°E	33	7.0	VII MMI	The quake was an under-sea quake and its epicenter was far from the land but, due to the tsunami which accompanied it, most of the southern part of the sea coast of Bali, Lombok, Sumbawa, and Sumba was damaged. In Kuta-Bali one person killed and 5 houses collapsed, 26 boats damaged or missing. In Lombok 20 persons killed,

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
								115 houses damaged, 132 boats missing or damaged. In Sumbawa 81 people killed, 53 people missing; more than 1,000 people lost their property; 63 houses, one school building, one mosque collapsed and the others were cracked. In the whole of Nusatenggara Island the quake resulted in 107 people killed, 54 people missing, 440 houses damage/collapsed, 467 boats missing or damaged, 5 school buildings collapsed and 3 teacher's house damaged.
159	Marapi, West Sumatra	1979 Apr 28	10h 29m 55.5s	0.7°N, 99.5°E	normal	5.7	V MMI	The quake caused cracks in some houses in Pinangsore - Sibolga. The shock also felt in Padang, Padangpanjang, Bukittinggi, Batusangkar and caused people to panic. Two days after, at midnight on April 30, the Marapi disaster occurred, because materials such as stones and soil crashed down from the top and slopes of the mountain. The materials from Marapi washed away everything and resulted in 64 people killed, 9 people missing, 193 houses collapsed, 42 bridges damaged, 138 dams and irrigation schemes destroyed, 34 cattle killed. The disaster might have been caused not only by the quake but also by the heavy rain in Marapi and the surrounding district.
160	Lombok, Nusa-tenggara	1979 May 30	17h 38m 53s	8.2°S, 115.9°E	25	6.1	VIII-IX MMI	In Tanjung, many houses and buildings collapsed, especially the poorly designed structures and old houses / buildings. Some people killed and injured. Other affected areas were Buyan, Gangga, Kediri, Cakranegara Narmada, where many buildings and houses were also seriously damaged and some even collapsed. Two mosques in Narmada and Cakranegara suffered moderate damage, cracks developed in their walls. The tower of the mosque in Kediri cracked. In Ampenan and Mataram the quake caused only slight damage to houses / buildings. I fact, the damage to houses

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
								/ buildings was due to old or poor construction.
161	Sentani, Jayapura	1979 Jul 23	14h 52m 53s	2.5ºS, 140.4ºE	normal	5.7	VII MMI	The shock caused damage to buildings and houses in Sentani.
162	Japen - Irian Jaya	1979 Sept 12	14h 17m 52.4s	1.8ºS, 136.1ºE	50	6.4	VII MMI	This earthquake killed 2 people and injured 5 people slightly in Japen and Jobi villages. Many houses, buildings, school buildings and clinics collapsed or were seriously damaged. The villages that experienced damage are Ansus, Papuma, Serui, Ariepie, Aromarea, Sarawandori, Serui Laut, Kabuaena, Borai, Menawi, Kointunai, Dawai, Randawaya and Warironi. All the above-mentioned are located in Japen District.
163	Karangasem, Bali	1979 Oct 20	09h 42m 9s	8.25ºS, 116.0ºE	normal	5.8	VI MMI	The quake caused moderate damage to buildings and houses in Karangasem, Ampenen, and Mataram on Lombok Island, eastward of Bali.
164	Tasikmalaya, West Java	1979 Nov 02	22h 53m 2.6s	8.6ºS, 107.8ºE	64	6.4	VII MMI	In Tasikmalaya and surroundings the quake caused 163 houses and buildings to collapse; 1,430 houses were seriously damaged; one meeting hall and 24 school buildings were damaged, 3 mosques collapsed and 29 were seriously damaged; 159 news-stand were severely damaged. In Garut most old and poorly constructed stone houses collapsed; many permanent houses had cracks in walls. 10 people killed, 12 seriously injured. Cracks in the ground in an east-west direction were observed. The quake was accompanied by a roaring sound from under the ground. Abnormal sea tides were observed 2 days before the quake occurred in Pameungpeuk.
165	Bengkulu, Sumatra	1979 Dec 15	07h 02m 37s	3.5ºS, 102.4ºE	25	6.0	VII-VIII-IX MMI	The quake caused damage in Kepahiang and Curup. No one killed or injured by this quake, but many houses and

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
								building seriously damage. In Kepahiang, more than 550 houses seriously damaged and in the Rejang Lebong area around 630 houses also seriously damaged; many others cracked on walls. Ground-slides and cracks were observed. Near Bengkulu, many houses were shifted from their foundations and water pipes were broken.
166	Karangasem, Bali	1979 Dec 18	03h 58m 26s	8.4°S, 115.8°E	28	5.0	VII-VIII MMI	In Karangasem this earthquake killed 5 persons, seriously injured 34 and slightly injured 250. Some houses collapsed, some were seriously damaged, many slightly damaged. In Abang, 17 people killed, 9 seriously injured, 300 slightly injured. Some houses collapsed; others seriously or slightly damaged. In Culik many houses collapsed. In Kubu, one person killed, 2 persons seriously injured and 18 slightly injured. Most buildings seriously or slightly damaged, but no building collapsed. In Bebandem, one person killed, 2 seriously injured and 4 slightly injured. Buildings and houses seriously or slightly damaged. Cracks in the road and land were observed along 0.5 km.
167	Manado, North Sulawesi	1980 Feb 22	11h 51m 46s	1.5°N, 124.65°E	normal	5.5	VI MMI	In Manado crack developed in some buildings and houses. No one reported killed or injured.
168	Tasikmalaya, West Java	1980 Apr 16	19h 18m 19s	8.25°S, 108.8°E	normal	6.4	V-VI MMI	In Singaparna many houses had cracks in walls but in Tasikmalaya itself only some houses were cracked. In Garut and surroundings, the poorly constructed houses had cracks in walls; also in the districts of Sukawening, Pasanggrahan, Jamberea, Caringin etc. many cracks developed in walls. In the Singajaya district 10 elementary school buildings slanted. The quake also caused cracks in houses in Cilacap, Central Java. The shocks were felt in Bandung at Intensity III.

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
169	Ambon, Maluku	1980 Aug 17	17h 01m 58s	3.7ºS, 128.5ºE	normal	5.4	V MMI	Cracks in walls developed in some houses in the city of Ambon. No more damage was observed.
170	Karanganyar, Central Java	1981 Jan 01	09h 09m 52s	7.7ºS, 111.0ºE	shallow	6.0	VI MMI	The quake shook Karanganyar and surroundings and caused slight damage to some houses.
171	Yogyakarta, Central Java	1981 Mar 14	06h 22m 35s	8.95ºS, 110.4ºE	normal	6.0	VII MMI	The shock was felt in Yogyakarta and caused small cracks in the walls of the Ambarukmo Hotel. No other buildings or houses damaged.
172	Sukabumi, West Java	1982 Feb 10	16h 17m 50.2s	7.0ºS, 106.9ºE	25	5.3	VII MMI	The quake was felt in some places in the Sukabumi and Bogor areas. The shock caused serious or slight damage to many houses and buildings and 4 people were injured. No loss of life.
173	Ruteng, Flores Island	1982 Aug 07	04h 40m 52.5s	8.35ºS, 120.35ºE	18	5.6	VI-VII MMI	In Ruteng the quake was strong enough to make people panic and run out from their houses. No building or house collapsed, but one hospital, one school building and one government office building and some houses were seriously damaged, and a microwave station building was slightly damaged. In Pagal, north of Ruteng, two school buildings, one church and 2 clinics were slightly damaged, cracks developed in walls and plaster fell. Cracks in the ground were observed.
174	Una-Una, Central Sulawesi	1982 Aug 24	00h 46m 34.7s	0.06ºN, 121.23ºE	5	4.7	VII MMI	A small island in Central Sulawesi Province was hit by a moderate earthquake. The walls of several houses fell. Cracks developed in walls and plaster fell. No deaths or injuries were reported.
175	Larantuka, Flores	1982 Dec 25	20h 28m 2.7s	8.4ºS, 123.04ºE	normal	5.9	VII-VIII MMI	The quake caused serious damage in Larantuka, Solor and on Adonara Island in east Nusatenggara Province. Hundreds of houses collapsed and thousands were slightly damaged, 13 people were killed, 17 injured and more than 400 slightly injured.

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
176	Ambon , Maluku	1983 Mar 12	09h 53m 36s	4.4°S, 128.05°E	25	5.8	VI MMI	The shock caused slight damage in Ambon. The quake was accompanied by a tsunami along the coast of Ambon.
177	Banda Aceh, North Sumatra	1983 Apr 04	09h 51m 13.9s	5.8°N, 93.27°E	51	6.6	VI MMI	The quake caused both serious and slight damage at Banda Aceh. The walls of school buildings collapsed and window panes were broken. Some of the government buildings, for example the Treasure building, TV station building, one room of the University building and the telephone office building, were damaged. In Meulaboh on the west coast of Aceh one building was slanted.
178	Toli-Toli, Central Sulawesi	1983 Oct 16	13h 32m 24.8s	1.48°N, 121.01°E	46	5.9	VI MMI	According to the news in the Kompas newspaper, on October 21, 1983 the quake was caused 20 houses in Toli-Toli to collapse and 15 aborigines' houses were seriously damaged.
179	Sangihe-Talaud-North Sulawesi	1983 Oct 23	05h 48m 44.4s	4.0°N, 126.6°E	118	4.9	V MMI	In the Sangihe-Talaud Islands cracks developed in the walls of buildings. No loss of life was reported.
180	Central Sulawesi	1983 Oct 25	08h 36m 19.4s	1.6°N, 120.8°E	50	6	VII MMI	The quake was strong enough to cause 2 people killed, 4 injured and 24 houses seriously damaged, of which 20 collapsed. The shock was also felt in Palu, the capital of Central Sulawesi Province.
181	Waingapu-Sumba-Nusa-tenggara	1983 Oct 31	11h 37m 54.5s	9.55°S, 119.09°E	179	6.5	V MMI	Some houses around Mauhau Airport in Waingapu had cracks in their walls. The quake was felt in Ujungpandang and Denpasar (Bali).
182	Mamuju, Central Sulawesi	1984 Jan 08	23h 24m 14.4s	2.94°S, 118.73°E	95	5.9	VII MMI	This quake killed 2 persons, seriously injured 5, and slightly injured 84. In the affected area 15 government office buildings, 23 government houses, 31 school buildings, on clinic, one news-stand seriously damaged and about 16 government office buildings, 12 government houses, 14 school buildings and two clinics slightly damaged. Besides

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
								this above-mentioned damage, 213 local people's houses, 4 shops, 18 mosques and one church were seriously damaged and 321 people's houses, 4 shops, 13 mosques and 1 church were slightly damaged. Ground-slumps were observed on Tapalang. The shock was also accompanied by a tsunami.
183	Pahae Jae-Tapanuli-North Sumatra	1984 Aug 27	13h 41m 25.5s	1.5°N, 98.94°E	53	4.8	VIII MMI	In Sarula, three elementary school buildings and one junior high school building collapsed, one senior high school building was seriously damaged and the school caretaker's house shifted 30 cm from its foundations. Cracks developed in the walls of the local authority office, post office building, and clinic. In Perdamaian and Selangkitang villages two elementary school buildings partly collapsed. Ground-slides were observed. On the road between Tarutung and Padangsidempuan cracks in the ground developed and sulphurous gas was emitted. In Pinangsori-Sibolga small cracks developed in the walls of houses. The quake was felt as far as Gunung Sitoli on Nias Island, Rantau Prapat and Balige (Toba Lake). According to the news, the shock was also felt along the west coast of Malaysia.
184	Tarutung	26-Apr-87	02:22:07.2	2.244°S, 98.866°E	11	5.9		In Tarutung-Lake Toba area, at least two people killed, 22 injured and more than 300 buildings damaged
185	Majalengka	6-Jul-90	07:16:20.4	6.904°S, 108.120°E	14	5.8		Affected in Kuningan-Majalengka-Sumedang area. At least 103 people injured and about 10,300 houses, mosques and public buildings damaged or destroyed
186	Flores	12-Dec-92	13:29:26.3	8.480°S, 121.896°E	28	7.8	V MMI at Larantuka IV MMI at	At least 2,500 people killed or missing in the Flores region, including 1,490 at Maumere and 700 on Babi. 2,103 people were missing, more than 500 people were injured and 90,000 were left homeless. Nineteen people killed and 130

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magnitude	Intensity	Description
							Waingapu, Sumba & Ujung Pandang II MMI at Kupang	houses destroyed on Kalaotoa. Severe damage, with approximately 90 percent of the buildings destroyed at Maumere by the earthquake and tsunami; 50-80% of the structures on Flores were damaged or destroyed. Damage also occurred on Sumba and Alor. The tsunami on Flores ran inland as much as 300 meters with wave heights of 25 meters. Landslides and ground cracks were reported at several locations on the island.
187	Halmahera	21-Jan-94	11:24:29.9	10.15°S, 127.733°E	20	6.2		Seven people killed, 40 injured and 550 houses damaged in the Kau area. Felt strongly at Ternate.
188	Liwa	16-Feb-94	0:07:43.8	5°S, 104.3°E	33	6.3	IX-X MMI	Liquefaction, 200 killed, 411 injured, 940 slight injured. 1,120 houses severely damaged.
189	Banyuwangi	4-Jun-94	04:06:59.8	10.362°S, 112.892°E	26	7.8		The earthquake caused tsunami. At least 75 killed, 17 missing and 213 houses destroyed
190	Serui	21-Nov-94	01:59:06.4	2.001°S, 135.931°E	24	5.7		Twenty-eight people injured and many buildings damaged at Serui, Yapen.
191	Pulau Obi	28-Jan-95	05:16:52.1	4.434°S, 134.476°E	22	6.2		No casualties. The earthquake caused damage to several buildings.
192	Dili	14-May-95	19:33:18.8	8.378°S, 125.127°E	11	6.2		Eleven people missing on Timor. Several houses destroyed by a local tsunami in the Dili area. Landslides occurred in the epicentral area. Affected in Dili, Maliana and Maubara.
193	Palu	20-May-95	05:30:06.4	1.021°S, 120.505°E	26	5.5		In Parigi, Palu and Poso, 38 people killed. 26 people injured and 115 houses damaged in the Parigi area.
194	Kerinci	7-Oct-95	1:09:45.9	2.1°S, 101.3°E	33	7	VII MMI	87 killed, 9240 houses heavily damaged
195	Biak	17-Feb-96	23:21:22.3	0.567°S, 135.840°E	19	8.1		The earthquake caused tsunami. At least 101 killed, 51 injured, 281 missing and 855 houses destroyed, 281 school buildings damaged
196	Pare-pare - Pinrang	28-Sep-97	8:38:28.6	3.91°S, 119.7°E	33	6	V-VI MMI	7 killed, 26 injured, 38 houses damaged in Pare-pare, 5 killed, 51 injured, 100 houses damaged in Pinrang

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
197	Pandegelang	22-Dec-99	21:14:57.6	7.21°S, 105.64°E	25	6	IV-V MMI	1 killed, 17 injured, 160 houses damaged
198	Banggai	4-May-00	11:21:16.2	1.65°S, 123.79°E	68	6.5	VI-VII MMI	40 killed, 258 injured, 23.000 houses damaged
199	Bengkulu	4-Jun-00	23:28:26.1	4.7°S, 102°E	33	7.3	V-VI MMI	90 killed, 803 injured, 1782 hurt. At least 13 school buildings, 21 public buildings, 11 health facilities, and 18,378 houses suffered damage.
200	Manokwari	10-Oct-02	21:28:25.8	1.511°S, 133.973°E	10	6.7		At least one person killed, 659 houses destroyed, one hospital severely damage, and 2,395 people evacuated.
201	Karangasem	2-Jan-04	03:59:31.9	8.310°S, 115.788°E	45	5.8	V-VI MMI	At least one person killed, 22 injured and 2,000 buildings damaged on Lombok. At least seven people injured and 4,000 buildings damaged on Bali. Felt (VI) at Karangasem, and (V) at Mataram
202	Nabire	6-Feb-04	06:05:02.8	3.615°S, 135.538°E	17	7	IV-VI MMI	This quake killed 39 people, 722 missing, and 2,500 were left homeless, 858 houses damaged or destroyed and nine bridges damaged in the epicentral area. The airport runway was damaged and power outages occurred in the Nabire area. Felt (VI) at Nabire, (V) at Enarotali, (IV) at Manokwari, and Tembagapura.
203	Padang Panjang	16-Feb-04	21:44:39.9	0.466°S, 100.655°E	56	5.1	II-V MMI	At least 5 people killed, 7 injured and more than 100 houses damaged in the Padangpanjang area. Felt (V) at Padangpanjang; (IV) at Batusangkar, Bukittingi and Padang; (II) at Pekanbaru
204	Alor	12-Nov-04	05:26:41.1	8.152°S, 124.868°E	10	7.5	V-VIII MMI	At least 33 people killed, 168,965 injured, 10,546 buildings damaged and/or destroyed. Landslides blocked roads in some areas. Felt (VIII) at Kalabahi, (V) as far away as Dili, East Timor.
205	Nabire	26-Nov-04	11:25:03.3	3.609°S, 135.404°E	10	7.1	VIII	At least 31 people killed, 228 injured, 328 buildings destroyed, 4,459 houses damaged, airport and seaport damaged and power outages occurred at Nabire.

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
206	Aceh	26-Dec-04	07:58:53.4	3.295°N, 95.982°E	30	9		The earthquake generated a large tsunami that travelled rapidly throughout the Indian Ocean, striking beachfront areas in many countries with catastrophic results in Indonesia, Thailand, Sri Lanka, India and Maldives, as well as other Asian and East African countries, killing approximately 150,000 people in the Indian Ocean region. The December 26, 2004 tsunami catastrophe was one of the worst natural disasters in modern history. Approximately $7 billion has been pledged for humanitarian emergency relief and reconstruction assistance of 127,400 houses to Tsunami affected areas in Aceh.
207	Palu	24-Jan-05	04:10:12.1	1.198°S, 119.933°E	11	6.3	III-V MMI	One person killed, four injured and at least 136 buildings damaged (V) in the Palu area. New hot springs formed in the Bobo area. Felt (V) in Palu, (III) at Parigi.
208	Nias & Simeulue	28-Mar-05	23:09:36	2.08°N, 97.01°E	30	8.7	II-IV MMI	At least 1,793 people killed, 12,556 missing, 157,793 evacuated, 105,549 houses severely damage, 2,739 school buildings damage, 294 health facilities damage, and 3,330 km road damage. Felt (IV) at Meulaboh and Sibolga; (III) at Banda Aceh, Sumatra. Felt at Medan, Sumatra. Felt (II) at Gelugor, Malaysia. Also felt at Bukit Mertajam, George Town, Kuala Lumpur, Sungai Ara and Tanjong Tokong, Malaysia.
209	Yogyakarta	27-May-06	05:53:58.9	7.961°S, 110.446°E	13	6.3		At least 5,749 people were killed, 38,568 were injured and as many as 600,000 people were displaced in the Bantul-Yogyakarta area. More than 127,000 houses were destroyed and an additional 451,000 were damaged in the area, with the total loss estimated at approximately 3.1 billion U.S. dollars.
210	Pangandaran	17-Jul-06	15:19:26.6	9.284°S, 107.419°E	20	7.7		The earthquake caused tsunami. At least 650 people died, 6,727 were homeless, 1,777 houses severely damage, and

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magnitude	Intensity	Description
								8 school buildings & 4 health facilities damaged. Affected in Ciamis, Tasikmalaya, Cilacap, and Kebumen.
211	North Sumatra	18-Dec-06	04:39:17.4	0.626°N, 99.859°E	30	5.8		Seven people killed, 100 injured and more than 680 homes damaged or destroyed in the Muarasipongi area.
212	West Sumatra (Solok-Padang)	6-Mar-07	10:49:38.9	0.493°S, 100.498°E	19	6.4	VIII	In Bukittinggi-Payakumbuh-Solok, Padang, at least 118 people killed, 255,627 were left homeless, 25,133 houses severe damaged or destroyed, 3,975 school buildings damaged.
213	Bengkulu	12-Sep-07	18:10:26.8	4.438°S, 101.367°E	34	8.5		At least 40 people killed. 56,390 buildings damaged or destroyed and roads damaged in Bengkulu and Sumatra Barat, 1,844 school buildings damaged.
214	Sumbawa (Dompu)	26-Nov-07	00:02:15.7	8.292°S, 118.370°E	20	6.5		6 people killed, hundreds injured and 3,620 houses destroyed (V) in the Bima-Dompu-Raba area, and 74 school buildings damaged.
215	Simeulue	20-Feb-08	15:08:30.5	2.768°N, 95.964°E	26	7.4		4 people killed, hundreds injured and 671 houses destroyed. 87 school buildings and 64 health facilities damaged.
216	Manokwari	4-Jan-09	07:33:40.2	0.691°S, 133.305°E	23	7.4		4 people killed, 25,079 were homeless, and 2,146 houses severely damage. 112 school buildings and 32 health facilities damaged.
217	West Java	2-Sep-09	14:55:01.0	7.782°S, 107.297°E	46	7		In Tasikmalaya, Garut, Pengalengan, Cikangkareng, at least 81 people killed, more than 1,297 injured, 196,153 displaced, 66,863 houses damaged, 5,600 school buildings damaged. Severe damage in Western Java, including at least 30 people killed and 27 missing due to a landslide at Cikangkareng.
218	West Sumatra	30-Sep-09	17:16:09.2	0.720°S, 99.867°E	81	7.6		At least 1,117 people killed, 1,214 injured, 181,665 buildings destroyed or damaged and about 451,000 people displaced in the Padang-Pariaman area.

No	Event	Local Date	Local Time	Epicenter	Depth (km)	Magni tude	Intensity	Description
219	Simeulue	7-Apr-10	05:15:01.5	2.383°N, 97.048°E	31	7.8	III-V MMI	At least 22 people injured, 18 houses damaged. Felt (V) at Meulaboh and Sibolga; (IV) at Banda Aceh and Medan; (III) at Padangsidempuan, Samosir and Tarutung
220	Mentawai	25-Oct-10	21:42:22.4	3.487°S, 100.082°E	20	7.8		The earthquake caused tsunami. At least 447 people died, 15,353 were homeless and 6 school buildings damaged.
221	Aceh	2-Jul-13	14:37:02	4.698°N, 96.687°E	10	6.1		In Aceh Tengah, Bener Meriah, at least 39 people killed, 400 injured, 3,000 houses damaged or destroyed.

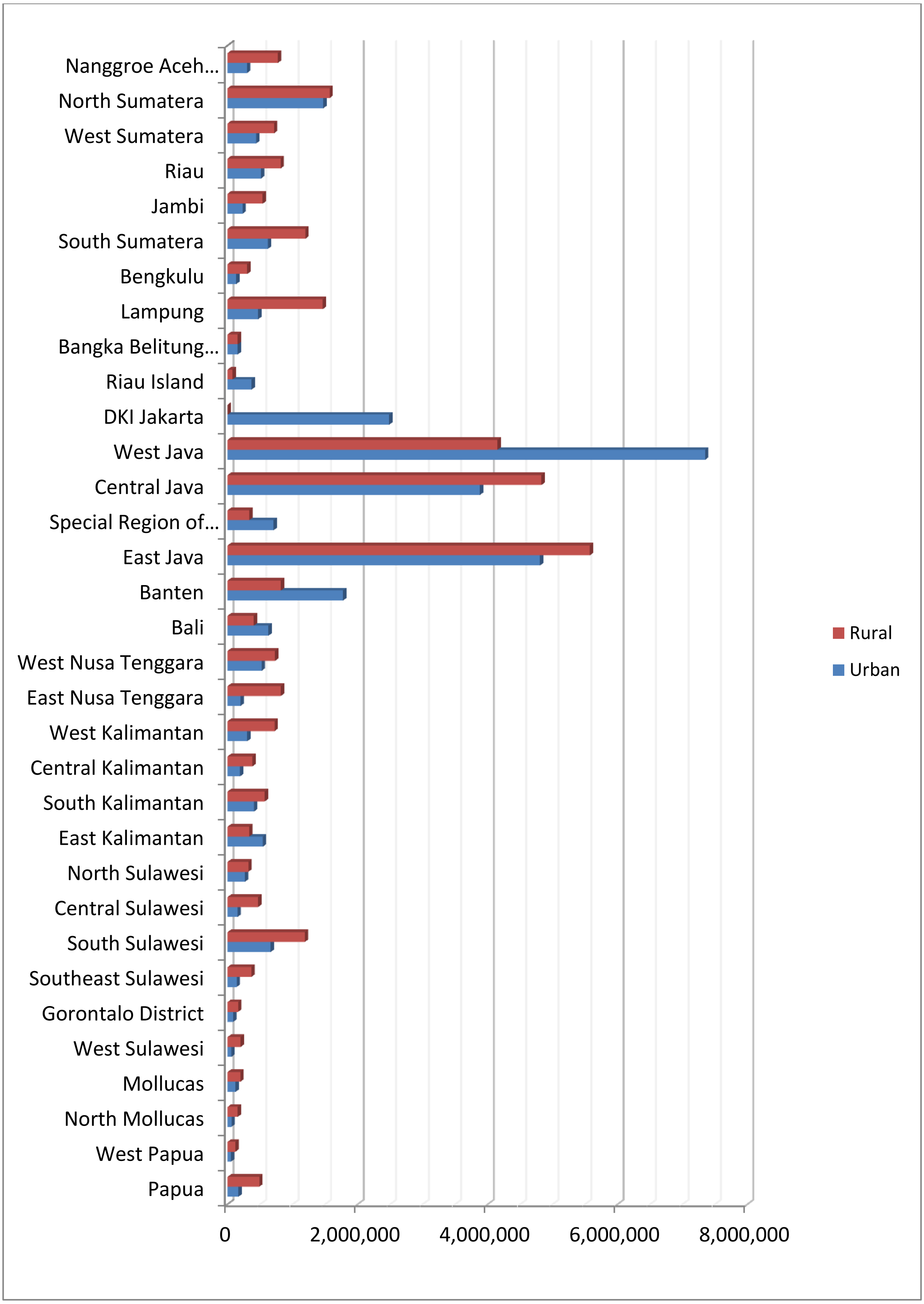
Source:

1. Southeast Asia Association of Seismology and Earthquake Engineering (SEASEE), Series on Seismology, Volume V - Indonesia, (Soetardjo, et al., 1985)
2. United States Geological Survey (USGS) http://earthquake.usgs.gov/
3. National Disaster Management Agency (Badan Nasional Penanggulangan Bencana – BNPB), www.bnpb.go.id

Appendix B Data of Houses in Urban and Rural Areas in Indonesia

No.	Province	Urban	Rural	Urban + Rural
1	Nanggroe Aceh Darussalam	296,334	769,989	1,066,323
2	North Sumatera	1,470,986	1,566,183	3,037,169
3	West Sumatera	439,093	711,964	1,151,057
4	Riau	511,303	813,688	1,324,991
5	Jambi	228,410	538,987	767,397
6	South Sumatera	619,318	1,194,117	1,813,435
7	Bengkulu	131,777	300,778	432,555
8	Lampung	469,215	1,462,164	1,931,379
9	Bangka Belitung Island	155,439	155,705	311,144
10	Riau Island	368,304	73,109	441,413
11	DKI Jakarta	2,484,103	0	2,484,103
12	West Java	7,345,846	4,146,932	11,492,778
13	Central Java	3,877,400	4,826,034	8,703,434
14	Special Region of Yogyakarta	706,149	331,701	1,037,850
15	East Java	4,805,404	5,572,661	10,378,065
16	Banten	1,773,499	817,822	2,591,321
17	Bali	627,246	400,925	1,028,171
18	West Nusa Tenggara	521,563	730,228	1,251,791
19	East Nusa Tenggara	195,975	817,859	1,013,834
20	West Kalimantan	301,671	721,309	1,022,980
21	Central Kalimantan	190,879	381,913	572,792
22	South Kalimantan	405,210	569,927	975,137
23	East Kalimantan	540,970	329,911	870,881
24	North Sulawesi	263,796	318,075	581,871
25	Central Sulawesi	150,372	470,031	620,403
26	South Sulawesi	661,734	1,185,944	1,847,678
27	Southeast Sulawesi	137,917	364,130	502,047
28	Gorontalo District	83,888	160,093	243,981
29	West Sulawesi	57,086	201,473	258,559
30	Mollucas	122,560	194,034	316,594
31	North Mollucas	59,087	155,205	214,292
32	West Papua	49,964	117,764	167,728
33	Papua	165,956	486,349	652,305
	Indonesia	**30,218,454**	**30,887,004**	**61,105,458**

Source: http://sp2010.bps.go.id/index.php/site/tabel?tid=299&wid=0



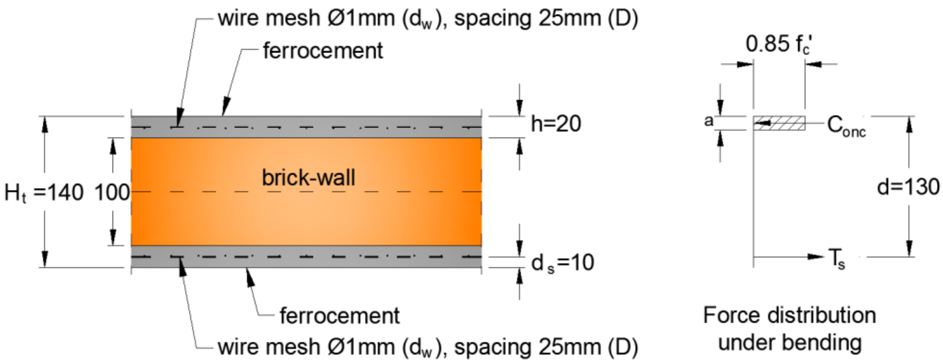
Number of Houses in Urban and Rural areas in Indonesia

Appendix C Data of School Buildings in Indonesia

No.	Province	Elementary School	Junior-High School
1	Nanggroe Aceh Darussalam	3,377	973
2	North Sumatera	9,366	2,365
3	West Sumatera	4,173	745
4	Riau	3,562	1,087
5	Jambi	2,459	691
6	South Sumatera	4,785	1,298
7	Bengkulu	1,329	412
8	Lampung	4,603	1,246
9	Bangka Belitung Island	782	193
10	Riau Island	826	277
11	DKI Jakarta	3,546	1,143
12	West Java	19,355	4,335
13	Central Java	19,400	3,324
14	Special Region of Yogyakarta	1,860	426
15	East Java	19,711	4,392
16	Banten	4,649	1,372
17	Bali	2,438	416
18	West Nusa Tenggara	3,078	847
19	East Nusa Tenggara	4,683	1,303
20	West Kalimantan	4,212	1,153
21	Central Kalimantan	2,514	723
22	South Kalimantan	2,892	604
23	East Kalimantan	2,220	702
24	North Sulawesi	2,151	668
25	Central Sulawesi	2,782	801
26	South Sulawesi	6,535	1,715
27	Southeast Sulawesi	2,286	699
28	Gorontalo District	916	337
29	West Sulawesi	1,348	356
30	Mollucas	1,720	570
31	North Mollucas	1,321	414
32	West Papua	961	273
33	Papua	2,327	521
	Indonesia	**148,167**	**36,381**

Source: http://www.sekolahdasar.net/2012/10/jumlah-sd-di-indonesia-ada-148361.html

Appendix D Bending Strength Analysis of Half-Brick-Thick Wall Panel (Simplified Method)



(a) *(b)*

Figure 141 – (a) Section of Half-Brick-Thick Wall Panel Strengthened using Wire Mesh; (b) Force Distribution under Bending

Analysis made based on (ACI Committee 549, 1999; Naaman, 2000)

<u>Data Mortar:</u>

Assumed mortar compressive strength: $f_c := 71.38 \frac{kgf}{cm^2}$

Total width of panel: $H_t := 14cm$

Width of one layer ferrocement: $h := 2cm$

Assumed length of panel for analysis $b := 1000mm$

Distance of wire mesh from the outer edge of ferrocement: $d_s := 1cm$

<u>Data Wire Mesh:</u>

Diameter of wire mesh: $d_w := 1mm$

Wire mesh spacing: $D := 25mm$

Number of wire mesh layer in one ferrocement layer: $n := 1$

Yield stress of wire mesh: $f_y := 6770 \frac{kgf}{cm^2}$

Elasticity modulus of wire mesh: $E_r := 2 \cdot 10^6 \cdot \frac{kgf}{cm^2}$

Efficiency factor in direction considered: $\eta := 0.5$ for welded square wire mesh

Volume fraction of one layer ferrocement: $V_f := n \cdot \left(\frac{1}{2 \cdot h \cdot D} \cdot \pi \cdot d_w^{\,2} \right)$

$V_f = 3.142 \times 10^{-3}$

Compressive area of mortar: $A_c := b \cdot h$ $\quad A_c = 2 \times 10^4\, mm^2$

Total equivalent area of mesh reinforcement in longitudinal direction: $A_{s1} := \eta \cdot V_f \cdot A_c$

$A_{s1} = 31.416\, mm^2$

Distance of tensile wire mesh from the top of compressive block: $d := H_t - d_s$ $\quad d = 130\, mm$

<u>Assumed all tensile reinforcement yield:</u>

Height of equivalent compressive block: $a := \frac{A_{s1} \cdot f_y}{0.85 \cdot f_c \cdot b}$ $\quad a = 3.505\, mm$

Check whether the compressive block do not exceed the width of one layer of ferrocement:

$$Check := \begin{cases} \text{"ok"} & \text{if } a < h \\ \text{"not ok"} & \text{otherwise} \end{cases}$$

Check = "ok"

Nominal tensile load resistance: $T_s := A_{s1} \cdot f_y$

$T_s = 2.127 \times 10^3\, kgf$

Moment nominal resistance: $M_n := T_s \cdot \left(d - \frac{a}{2} \right)$

$M_n = 272.764\, kgf \cdot m$

Strength reduction factor for bending: $\phi := 0.9$

The ultimate bending moment strength: $\phi \cdot M_n = 245.487\, kgf \cdot m$

Appendix E Ferrocement Properties Calculation

Ferrocement consists of mortar with 2cm thickness (*h*) and single wire mesh with 1mm diameter (d_w) spaced at 25mm (*D*) in both directions.

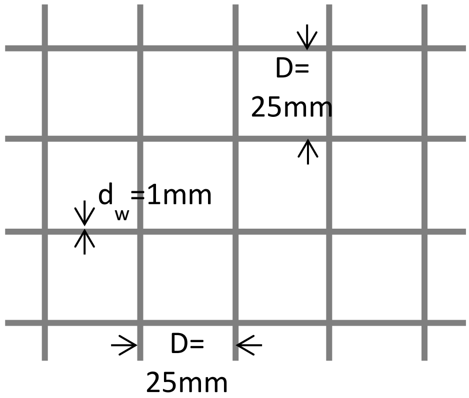


Figure 142 – Square Meshes for Ferrocement Reinforcement

The material properties of ferrocement are calculated based on (ACI Committee 549, 1999; Naaman, 2000; Bangladesh National Building Code, 2012) as follow:

- Volume fraction of mesh in longitudinal direction:

$$V_{rL} = \frac{N\pi d_w^2}{4h}\frac{1}{D}$$
$$= \frac{1\pi(1)^2}{4.\,20}\frac{1}{25}$$
$$V_{rL} = 0.001571$$

- Volume fraction of mortar:

$$V_m = 1 - V_{rL}$$
$$= 1 - 0.001571$$
$$V_m = 0.998$$

- Elasticity modulus of ferrocement:

$$E_c = E_m V_m + E_r V_{rL}$$
$$= (71{,}380)\,.\,(0.998) + (2{,}000{,}000)\,.\,(0.001571)$$
$$E_c = 74{,}470\ kg/cm^2$$

- Tensile stress of ferrocement:

$$\sigma_{cy} = \eta_o V_r \sigma_{ry}$$
$$= (0.5)\,2(0.001571)(6770)$$
$$\sigma_{cy} = 10.64\ kg/cm^2$$

Notations:

d_w = diameter of wire mesh (= 1mm)

h = thickness of ferrocement element (= 20mm)

D = distance center to center between wires (= 25mm)

E_m = elasticity modulus of mortar matrix (= 71,380kg/cm^2)

E_r = elasticity modulus of reinforcement (= 2,000,000kg/cm^2)

N = number of layers of mesh (= 1)

V_m = volume fraction of mortar matrix

V_{rL} = volume fraction of reinforcement in longitudinal direction

σ_{cy} = tensile stress in composite corresponding to yielding of reinforcement (σ_{ry})

σ_{ry} = yield stress of reinforcement (= 6,770kg/cm^2)

η_0 = efficiency factor in direction considered (= 0.5 for welded square wire mesh)

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